CONTROL EQUIPMENT

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Glenfield
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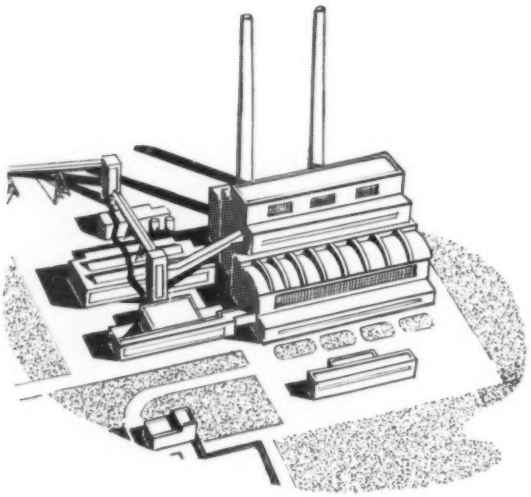
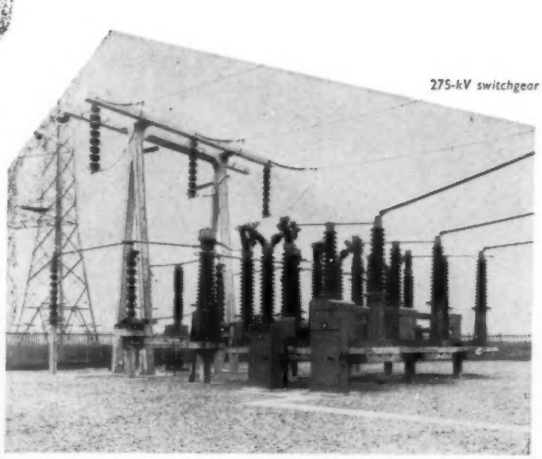
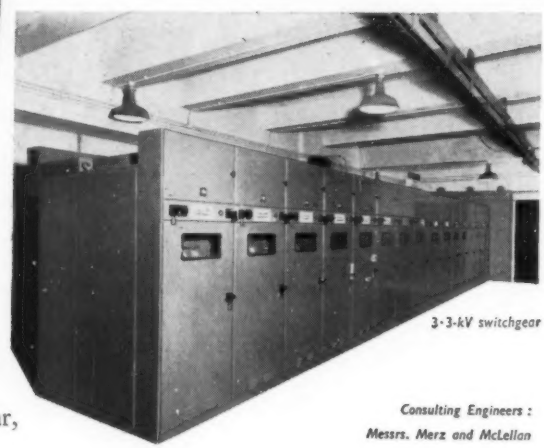


Blyth A
GENERATING STATION



275-kV switchgear

Main control-room



3·3-kV switchgear

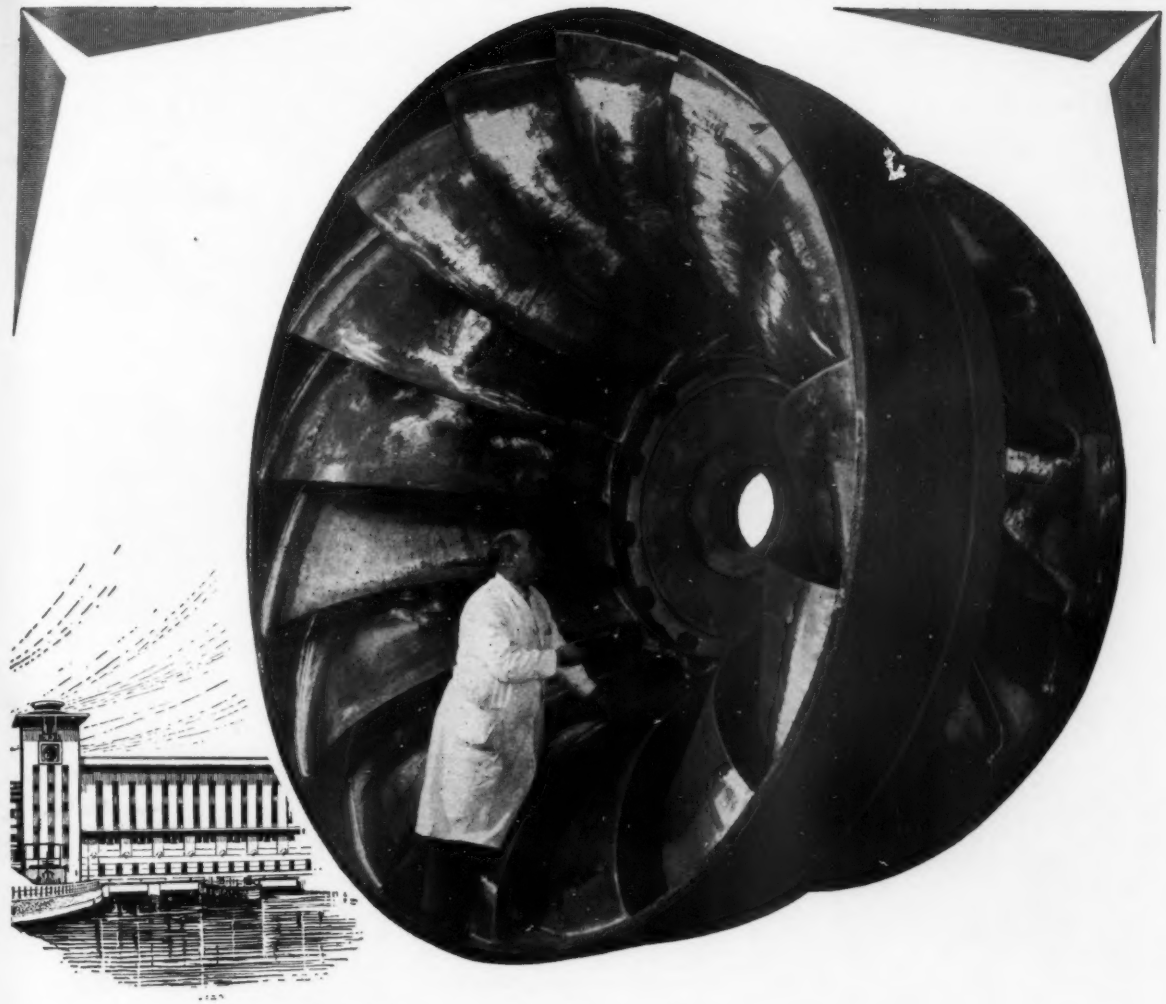
Reyrolle supplied the 275-kV air-blast switchgear, 66-kV small-oil-volume switchgear, 3·3-kV air-break auxilliaries switchgear, and 415-volt distribution gear together with associated control-equipment.

Consulting Engineers :
Messrs. Merz and McLellan

Photographs by courtesy of C.E.G.B., North Eastern Division

Reyrolle

A. REYROLLE & COMPANY LIMITED - HEBBURN - COUNTY DURHAM - ENGLAND



India again chooses...

Five 'ENGLISH ELECTRIC' water turbine generating sets have been delivered to the Rihand Power Station. These sets were ordered by the Government of the State of Uttar Pradesh, India, following the commissioning of the Khatima hydro-electric station for which the Company supplied and erected the entire plant and equipment.

For the Rihand Station, the five turbines are of the Francis type with a head range of 143/250 ft. and a rating at 225 ft. of 77,000 h.p., speed 150 r.p.m. The umbrella type alternators each generate 61,000 kVA, 0·9 p.f. at 11 kV.

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Hydro-electric Department, Dunnings Bridge Road, Netherton, Bootle, 10

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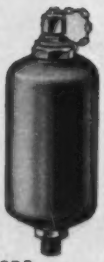
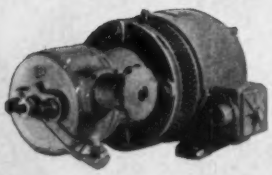


HYDRAULIC ACCUMULATORS



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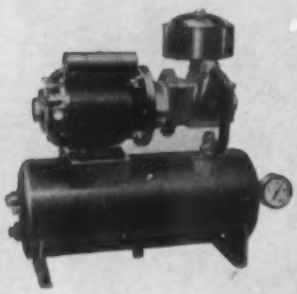

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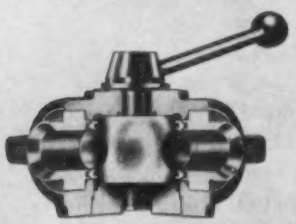
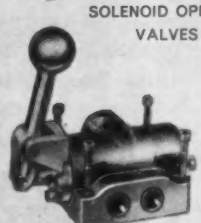
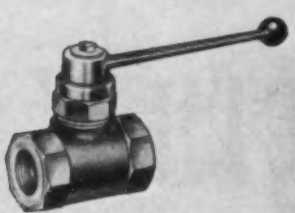


MULTI-WAY HIGH PRESSURE VALVES

SOLENOID OPERATED VALVES



PNEUMATIC AND HYDRAULIC VALVES AND CYLINDERS



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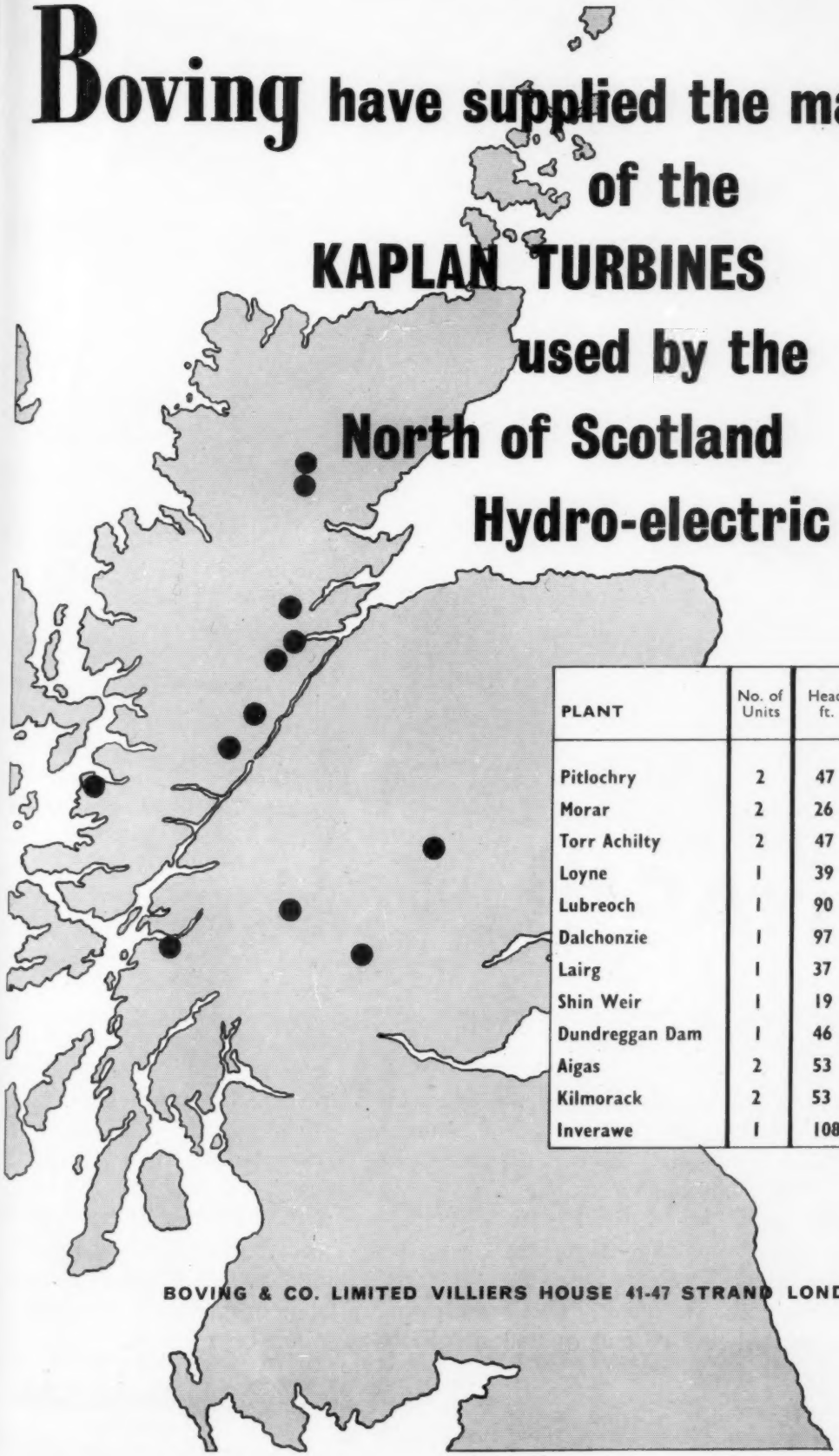

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Morar	2	26	333	730
Torr Achilty	2	47	167	10,900
Loyne	1	39	434	735
Lubreoch	1	90	334	5,600
Dalchonzie	1	97	334	5,600
Lairg	1	37	167	4,906
Shin Weir	1	19	434	145
Dundreggan Dam	1	46	1020	225
Aigas	2	53	187	14,000
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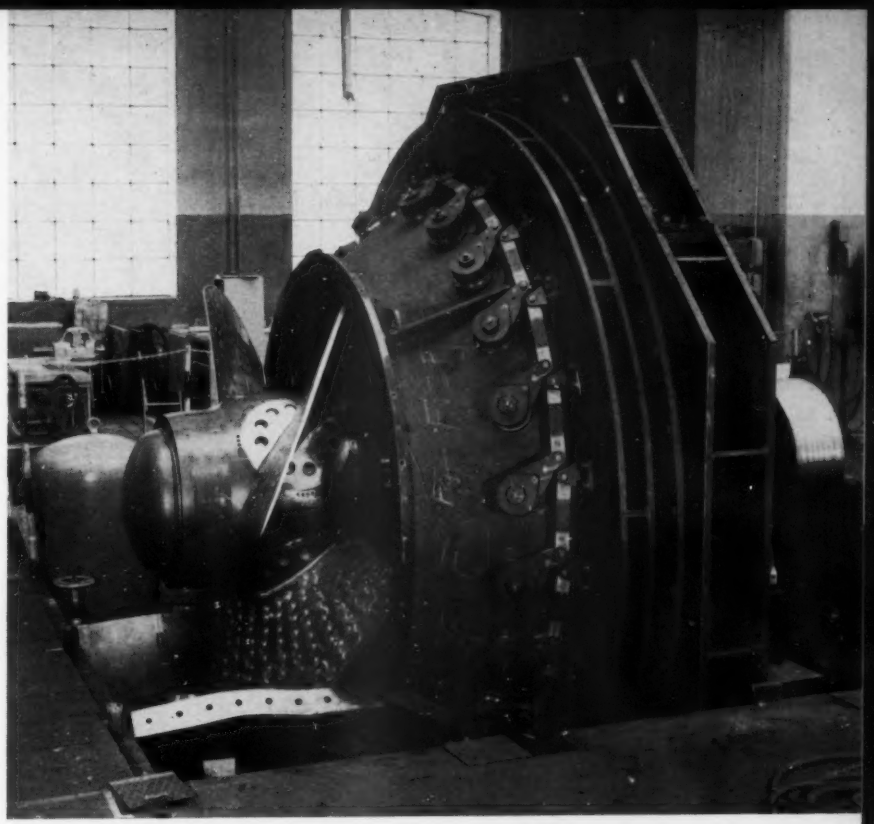
Sweden's first tubular turbine

now in service at Skogsforsen power station (Yngeredsfors Kraft AB)



KMW and ASEA have now completed the first tubular turbine unit in Sweden. The unit comprises in principle a horizontal Kaplan turbine with a direct-coupled synchronous alternator. The alternator is arranged concentrically in the waterway, so that the stator casing forms an extension of the internal wetted perimeter of the turbine. This setting provides a straight waterway with essentially axial flow.
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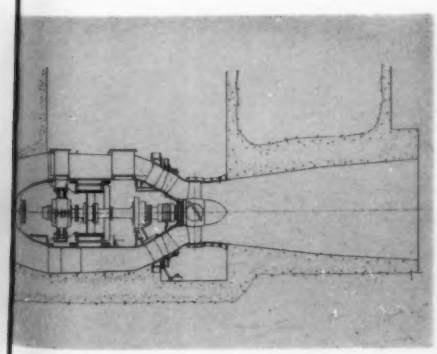

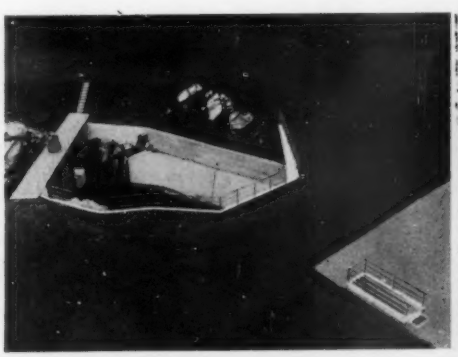

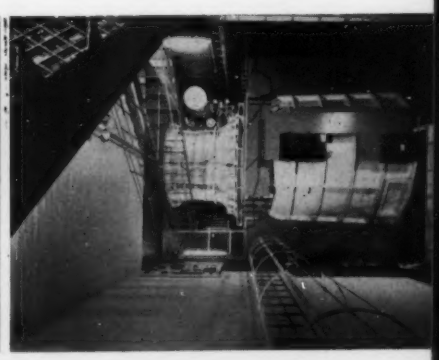

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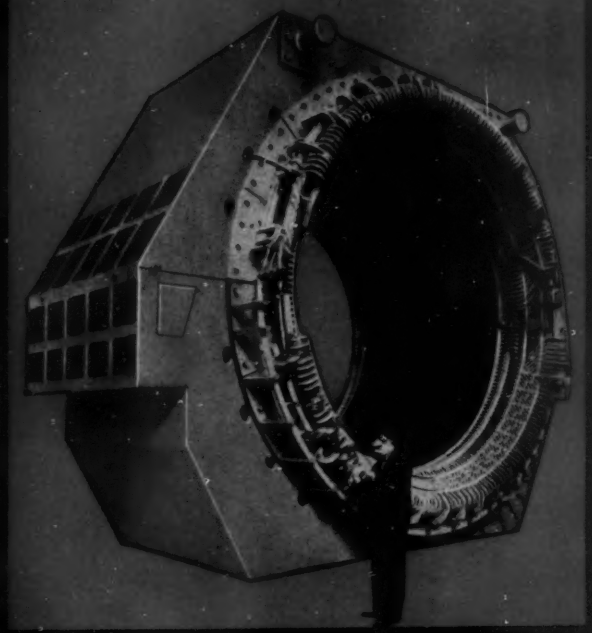

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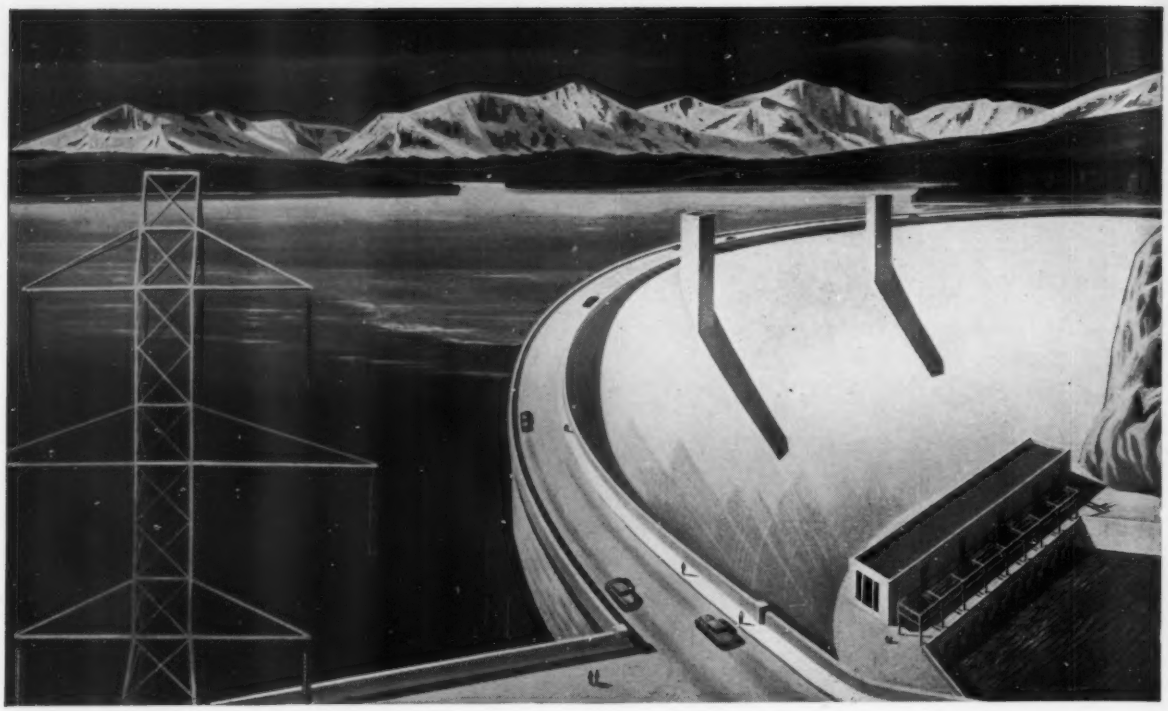

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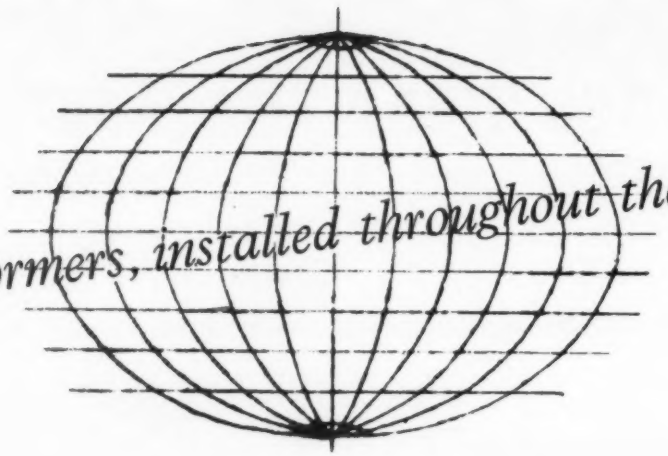

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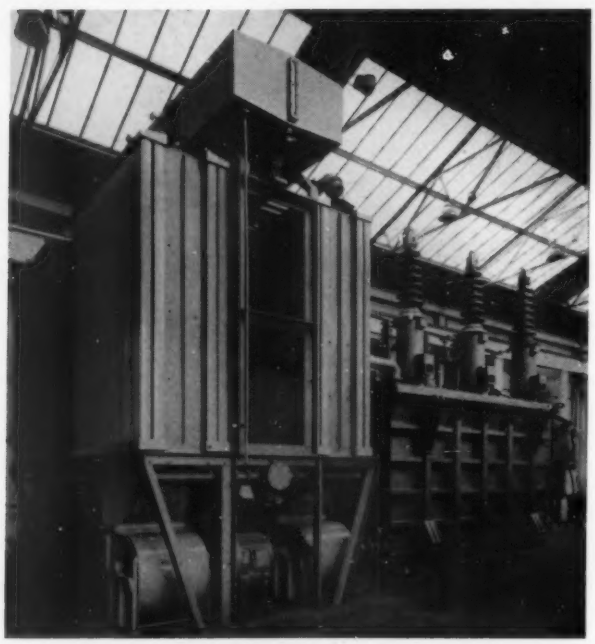

132 kV

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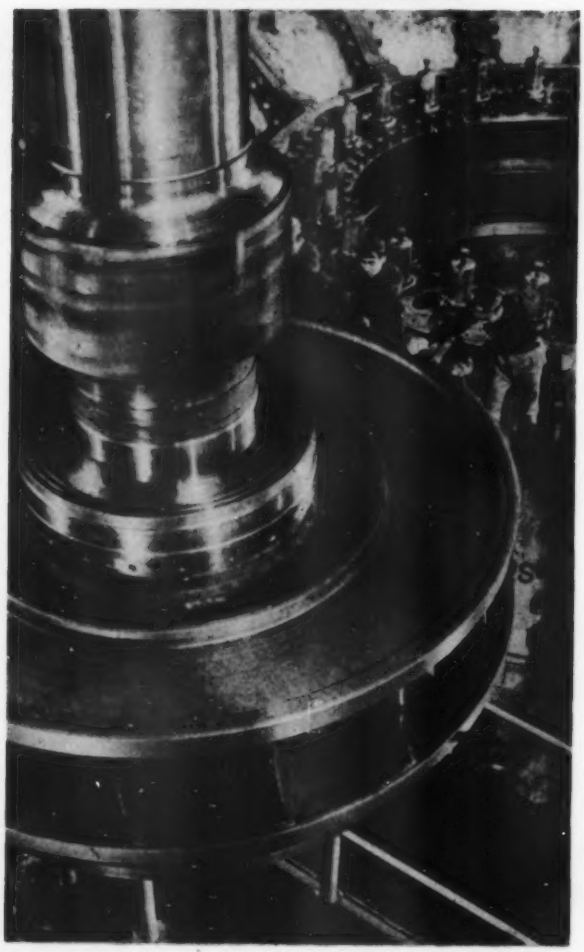

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The 137,000 KW Francis turbine, seen at left, to be coupled with this generator is also a Toshiba product. The runner diameter of this turbine is 4,000 mm, and ranks among the biggest in the world.

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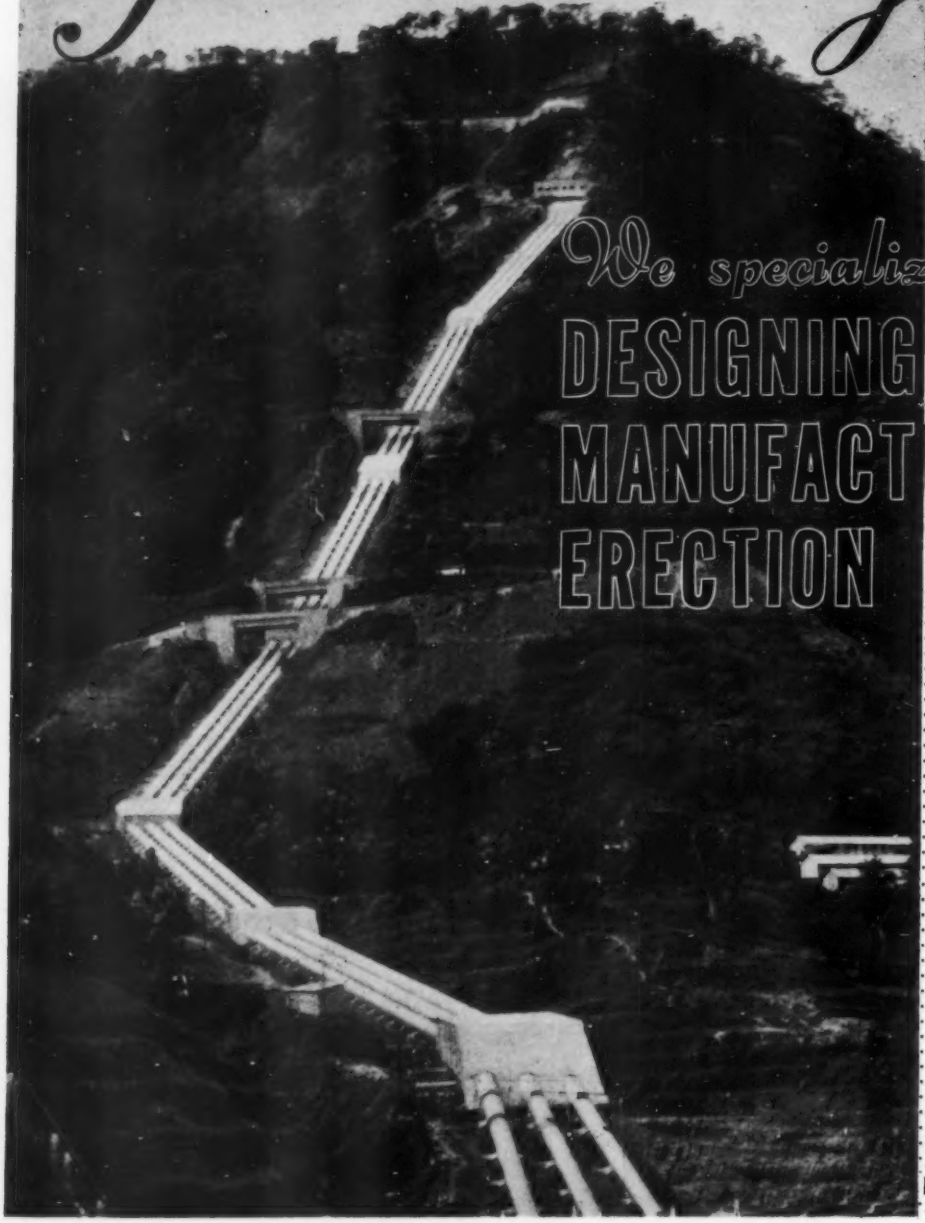

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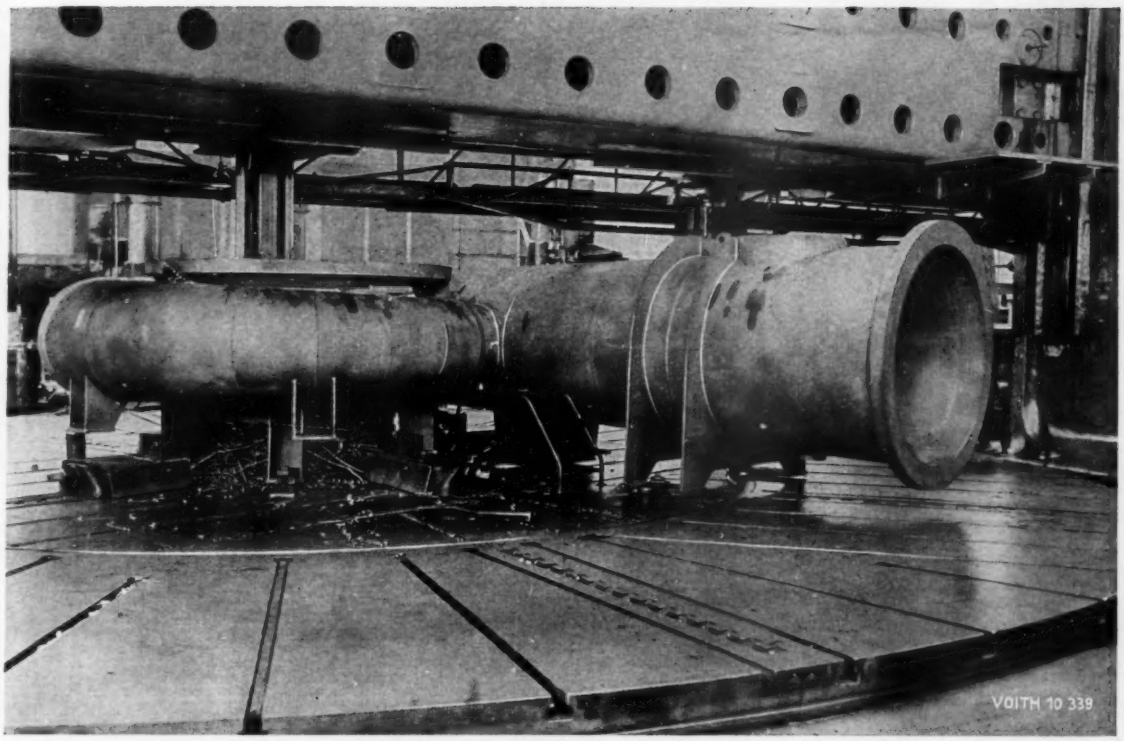

Hydraulic Turbines for the new capital of Brazil

The Voith turbines under construction for Brasilia, two vertical-shaft Francis spiral turbines, will develop a total output of 25,000 b.h.p. at 514 rpm and 105 m net head. The order also includes the supply of three hydraulically actuated butterfly valves, ins. dia. 1,400 mm. The photo shows one of the spiral casings of the Brasilia turbines on our large vertical boring mill. The two Brasilia turbines and the four Três-Marias Kaplan turbines, each rated at 100,000 b. h. p. under 55 m net head, are another Voith contribution to the harnessing of the Brazilian water resources.

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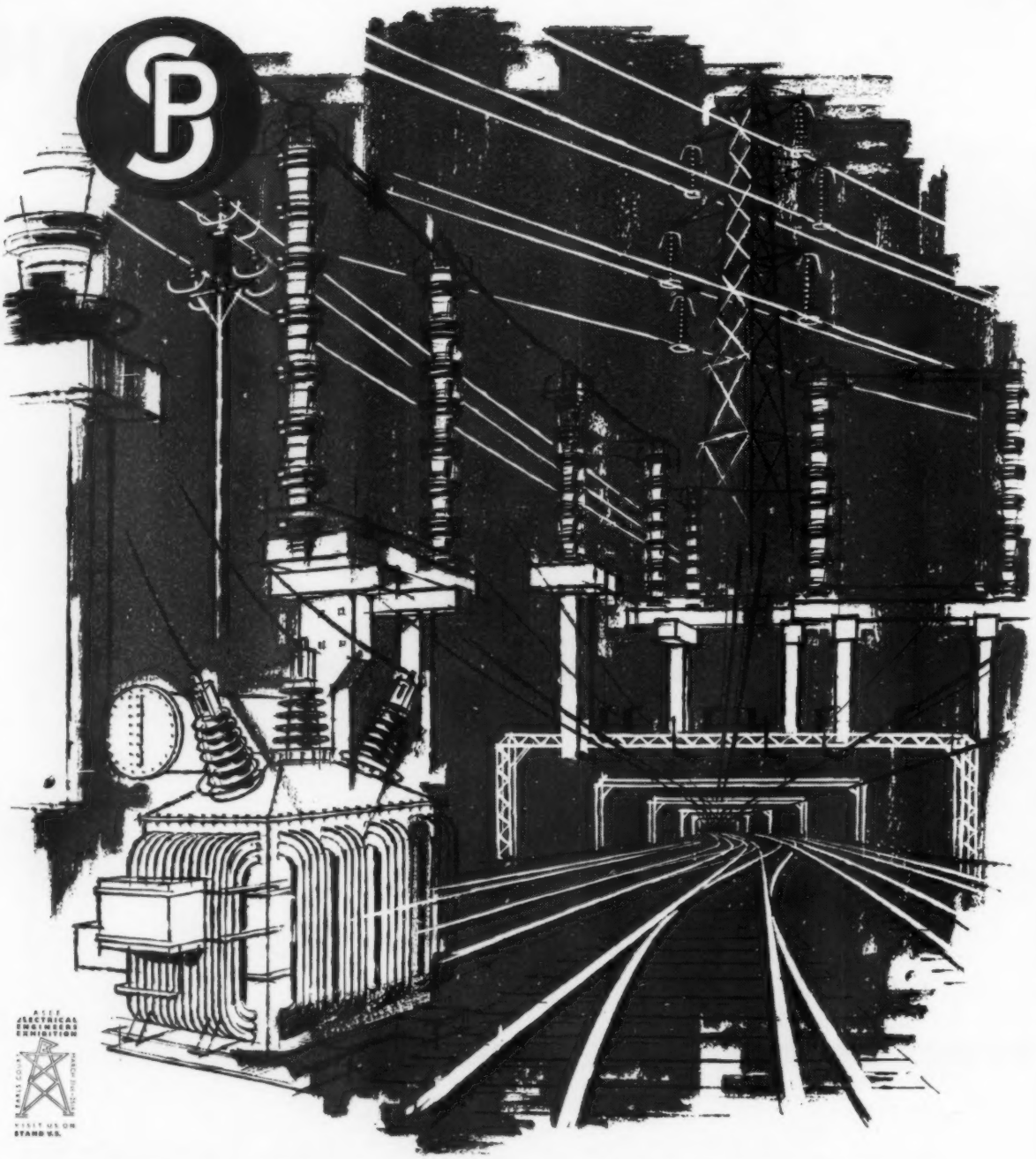

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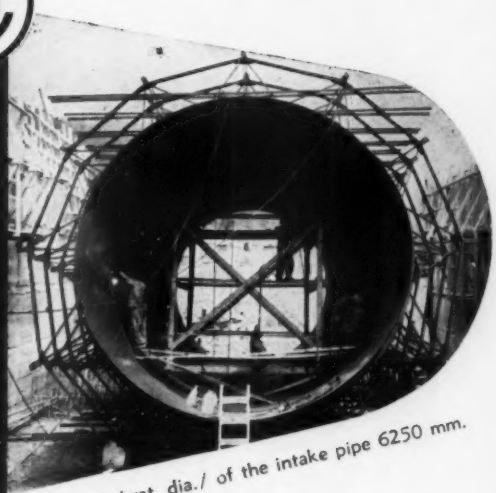
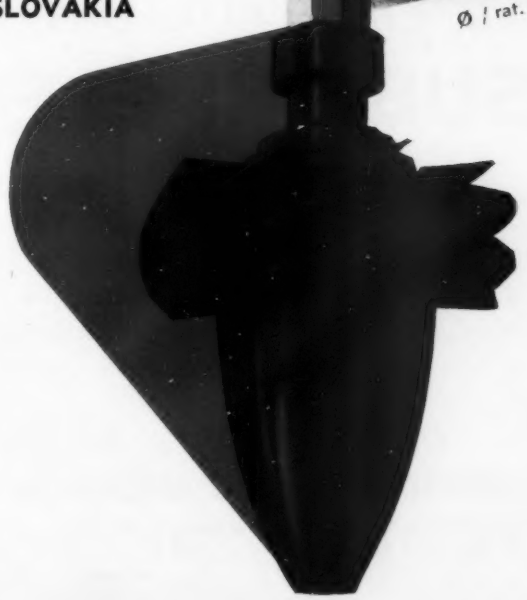
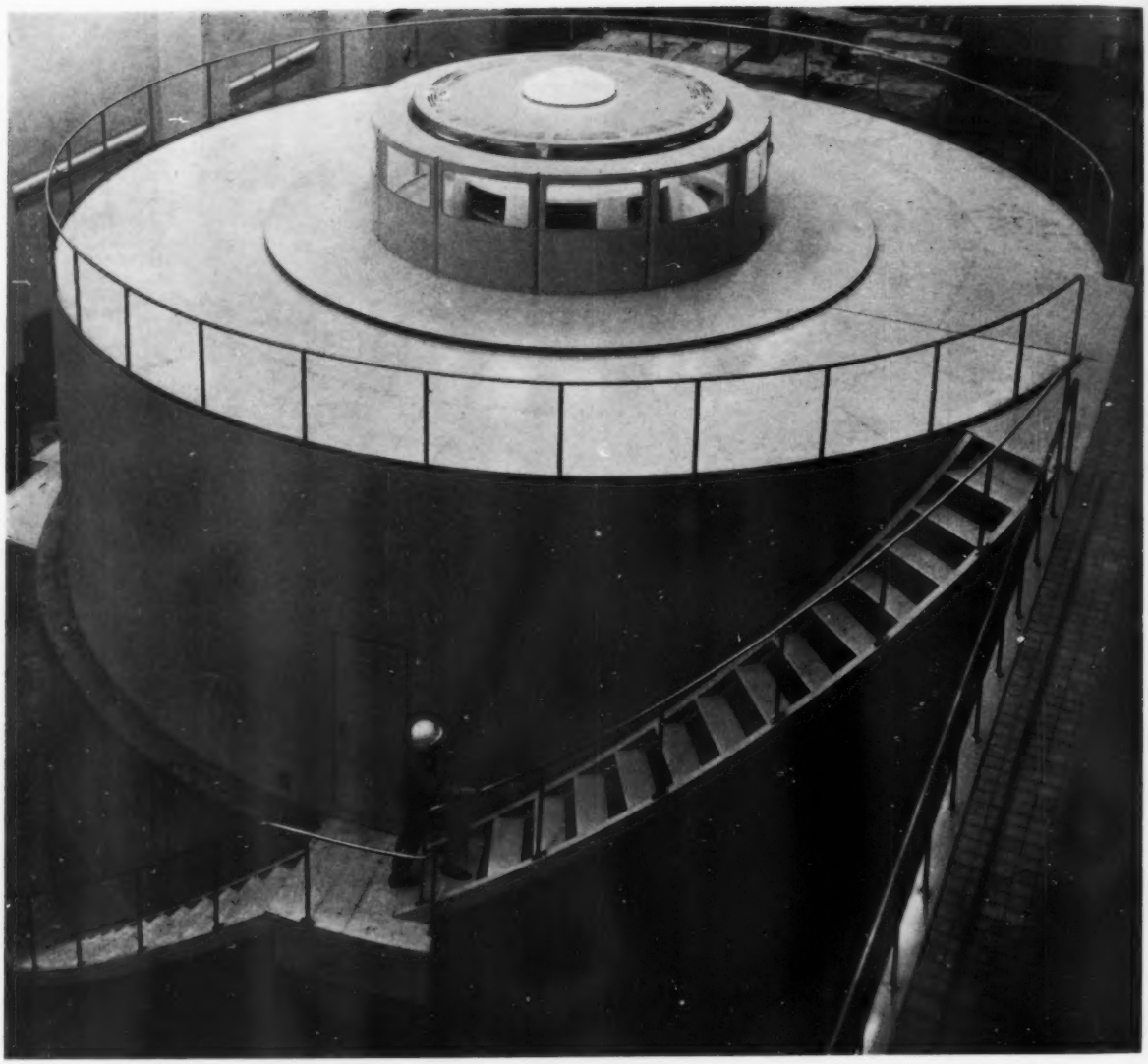


Ø / rat. dia./ of the intake pipe 6250 mm.



105,000 k VA Water-Wheel Generators



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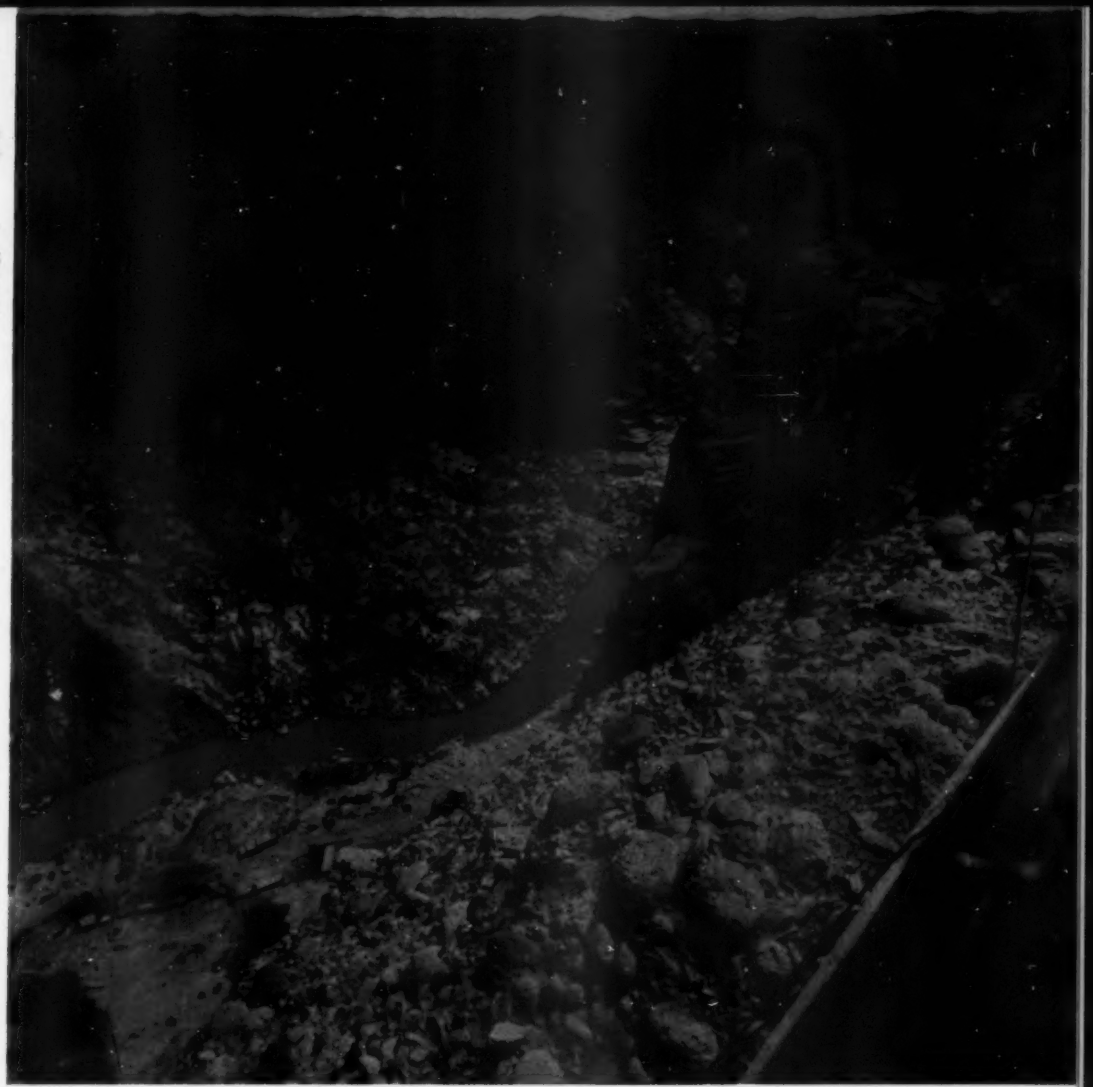

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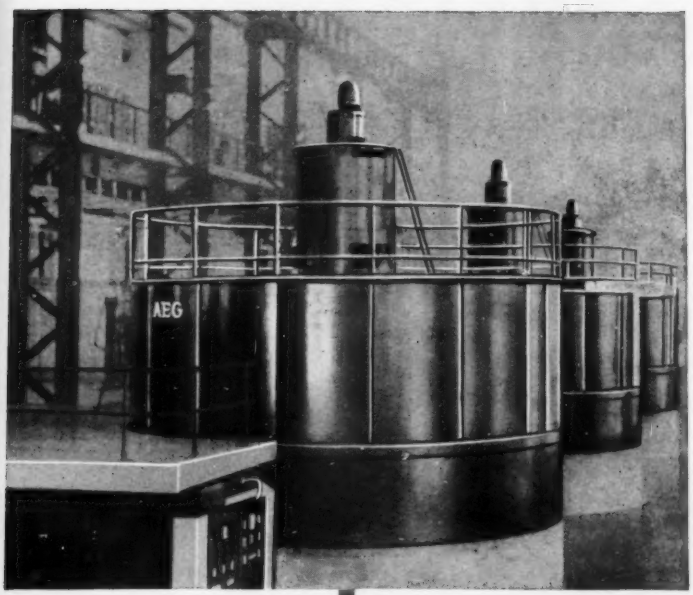

Three 41 200 kVA generators at Periyar Hydro-Electric Station, India

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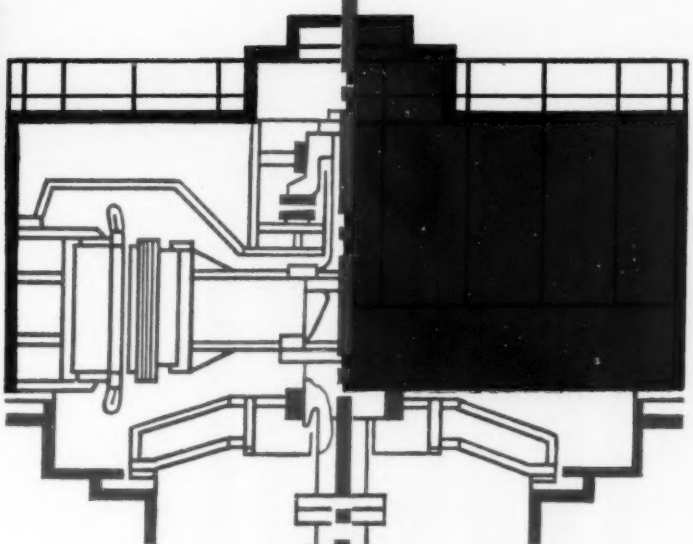
AEG waterwheel alternators



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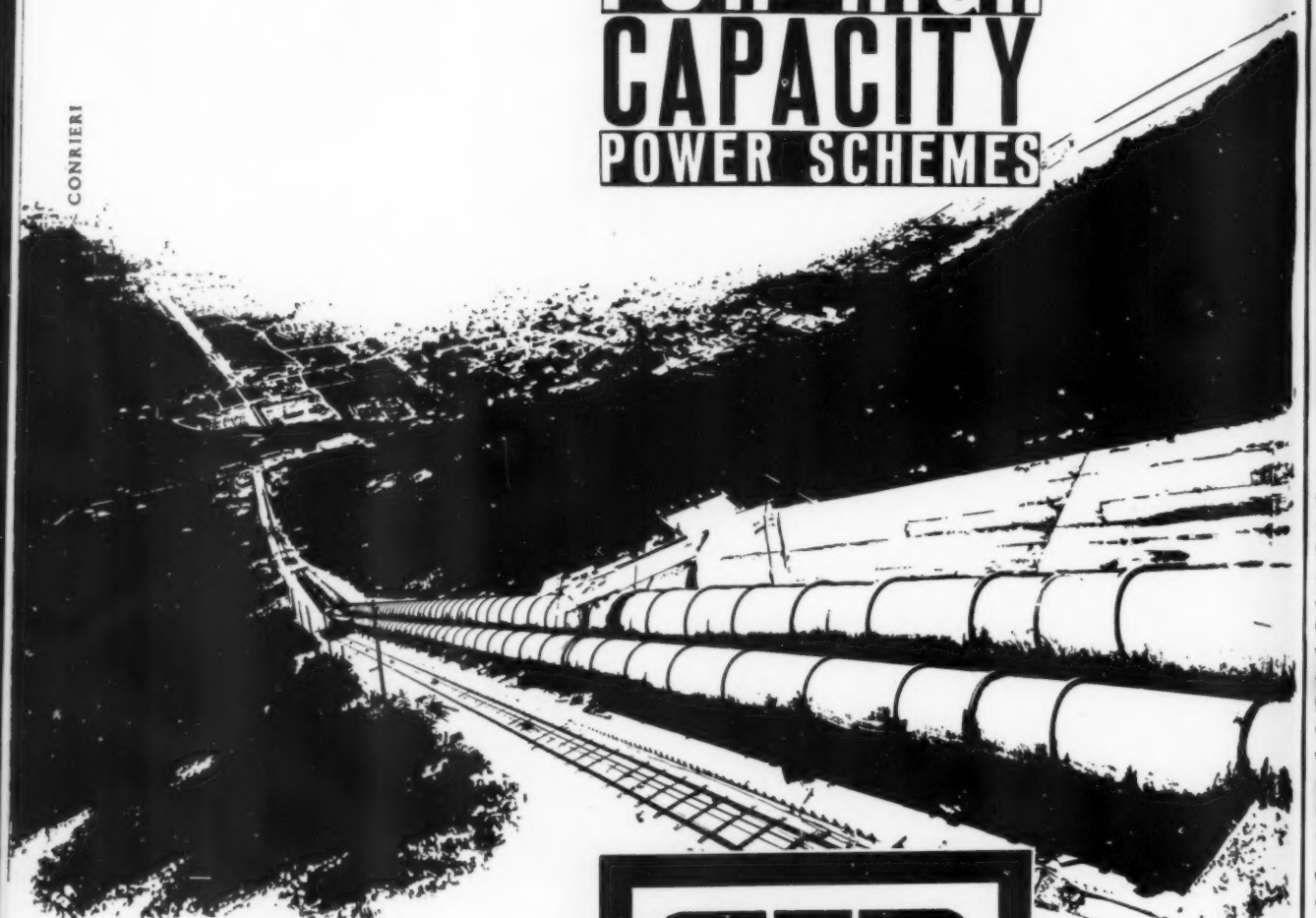

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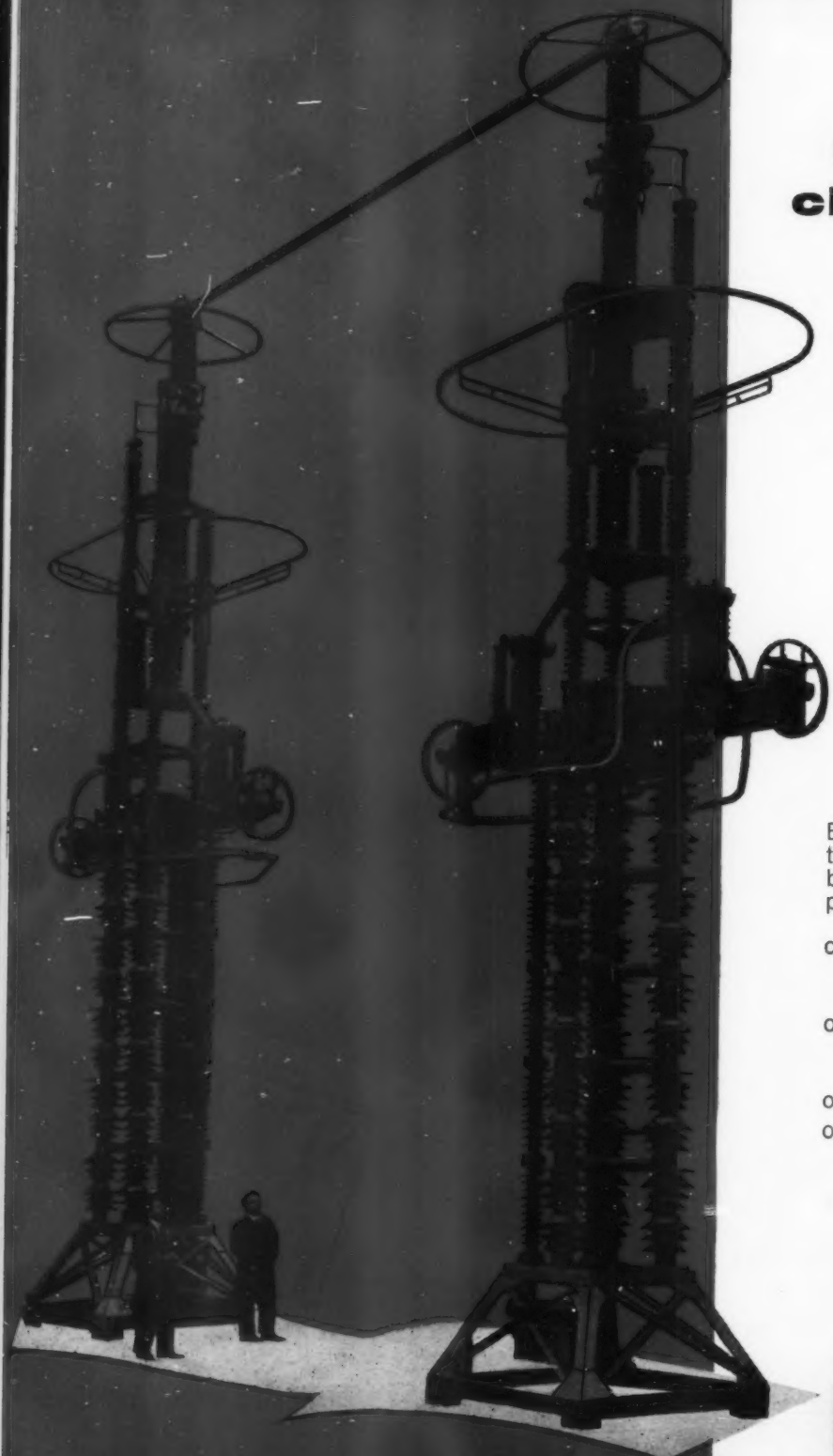

One of the poles of a series of 525 kV, 25,000 MVA, AE 17 air blast circuit breakers, for the equipment of the U.R.S.S network.

the air blast circuit breaker

type AE

A range of breakers from 70 kV to 525 kV
Breaking capacity: 30,000 amp. rms.

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- o Negligible burning of contacts.

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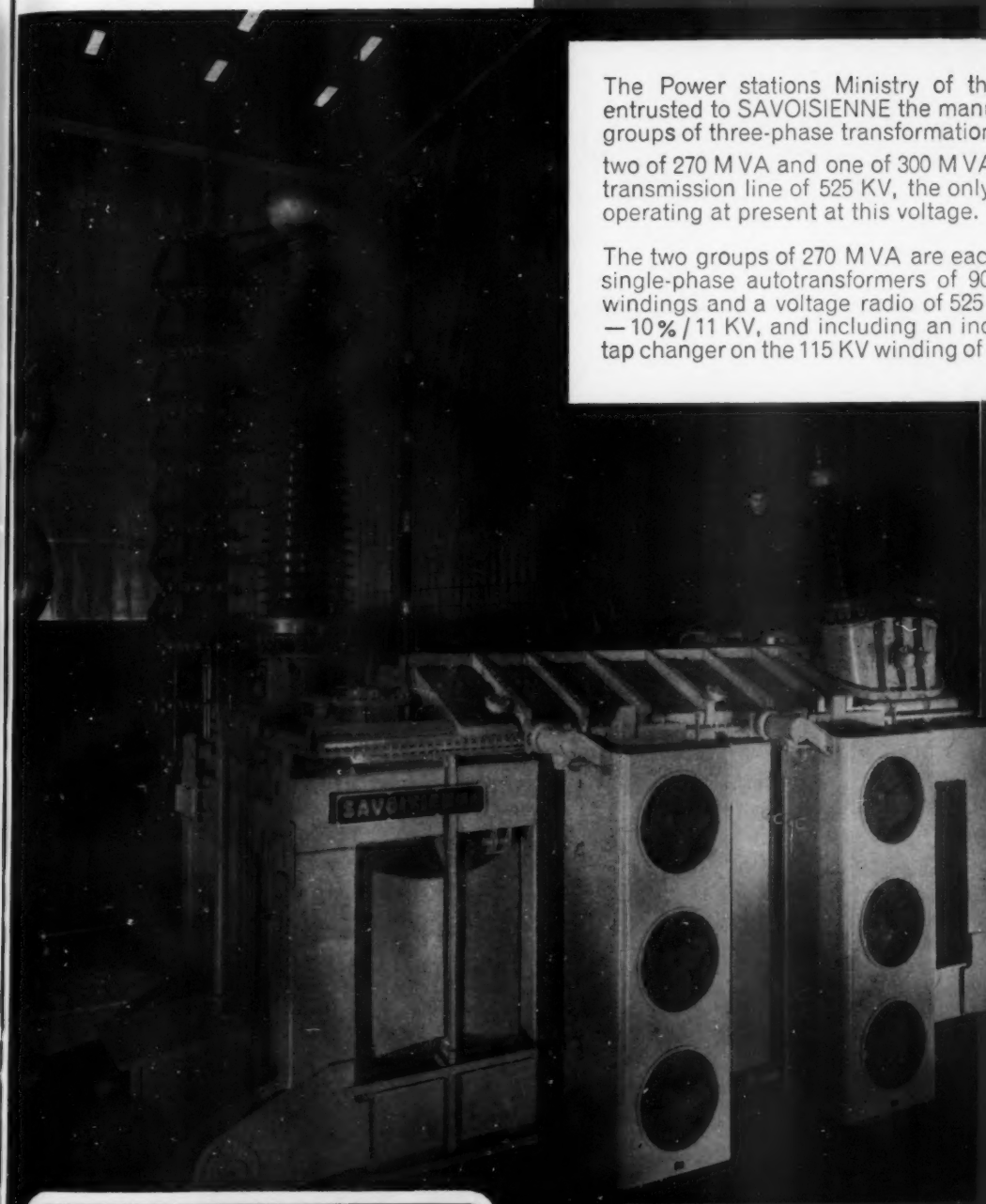
GE TRANSMISSION LINE

525 KV



The Power stations Ministry of the U. S. S. R. has entrusted to SAVOISIENNE the manufacturing of three groups of three-phase transformation :

two of 270 M VA and one of 300 M VA, intended for the transmission line of 525 KV, the only one in the world operating at present at this voltage.

The two groups of 270 M VA are each formed by three single-phase autotransformers of 90 M VA with three windings and a voltage radio of 525 V 3 / 115 V 3 + or — 10 % / 11 KV, and including an incorporated on-load tap changer on the 115 KV winding of + or — 10 % steps.

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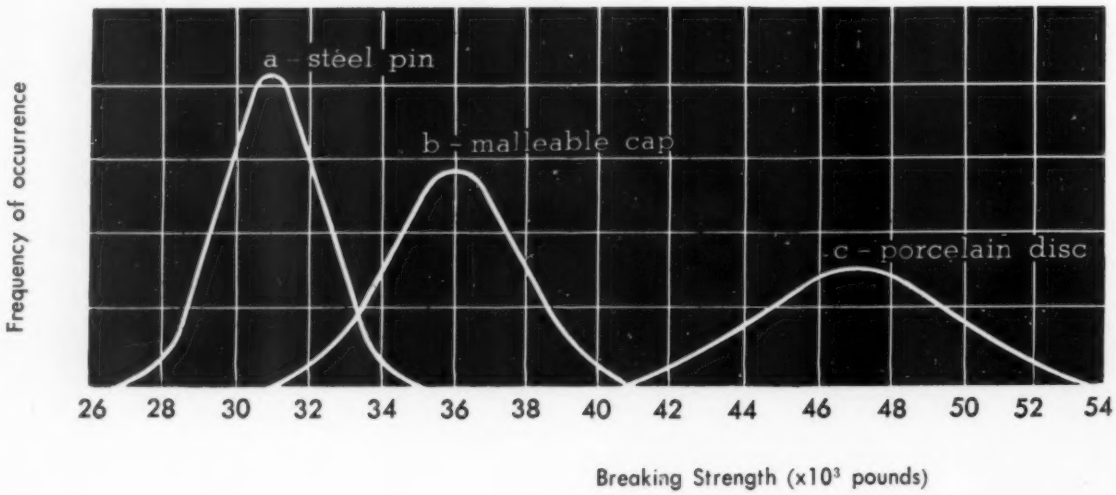
a - steel pin
b - malleable cap
c - porcelain disc
Frequency of occurrence
26 28 30 32 34 36 38 40 42 44 46 48 50 52 54
Breaking Strength (x10³ pounds)

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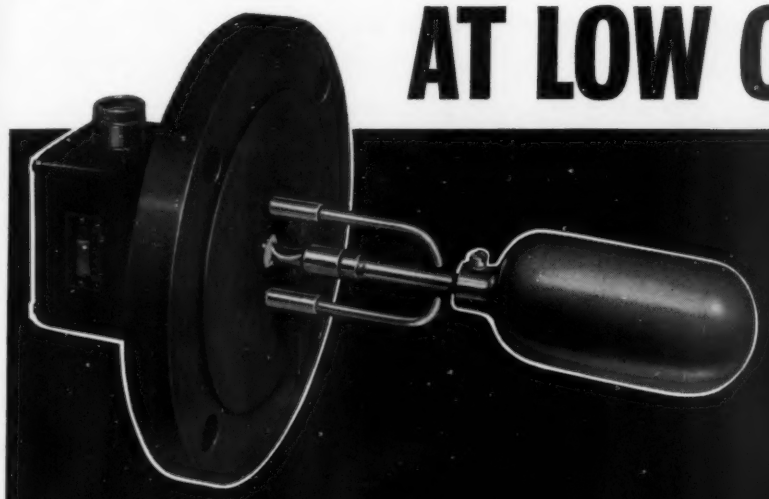

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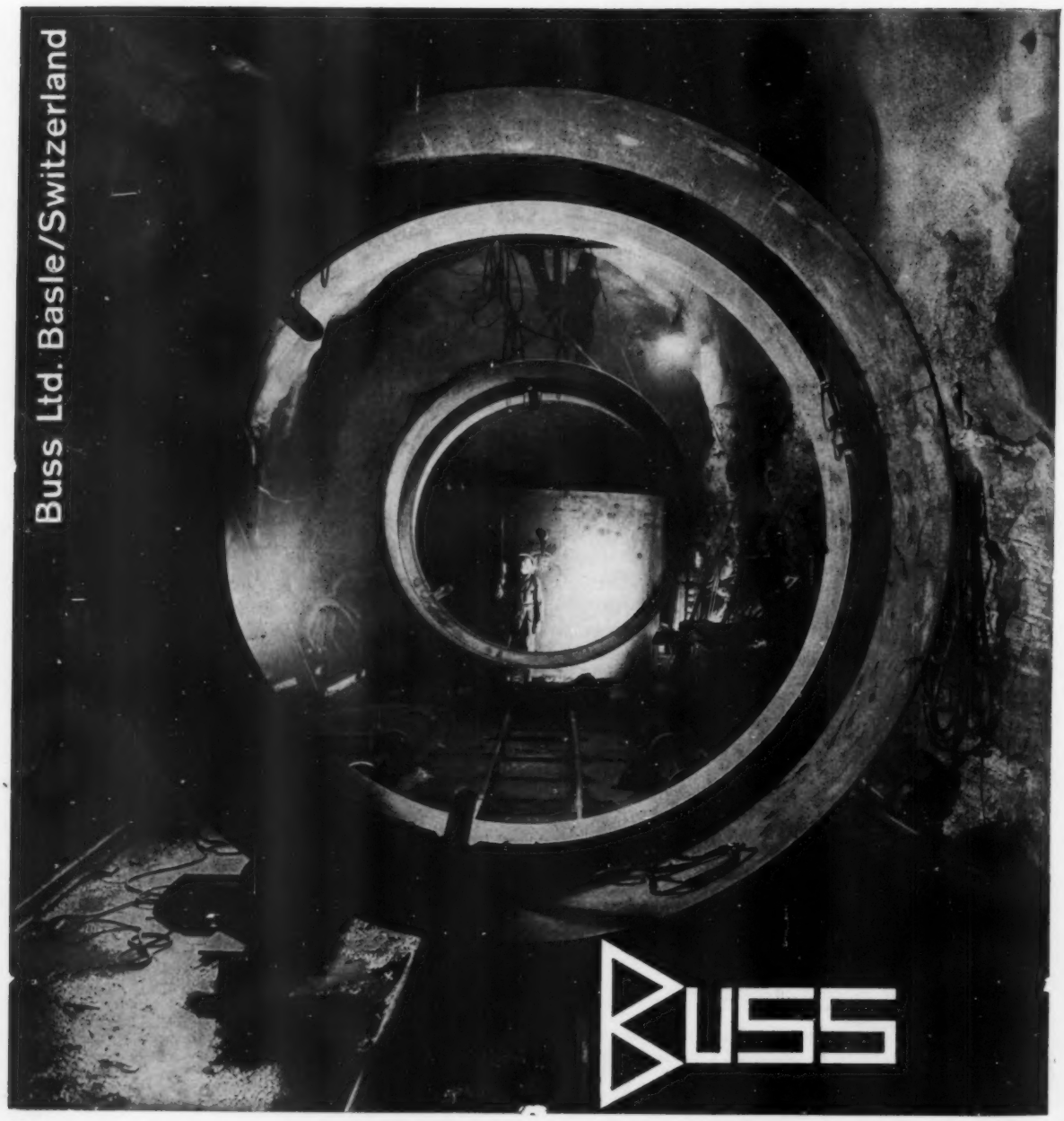

the bigger the better

In the 2" bore size, the face to face dimension of a Cockburn-Rockwell Butterfly Valve is 1¾". With a 17½" bore, the valve is 4" face to face. In the 35½" size, the economy in space is even more marked—only 8½" of the pipe run is occupied by the valve. Cockburn-Rockwell Butterfly Valves are the most compact form of pipeline control, and this feature becomes more and more appreciable as the size increases. Valves are currently being manufactured with ports up to 60" bore.

The Valve illustrated, 42" diameter, is one of a large order supplied to the Greek Water Board.

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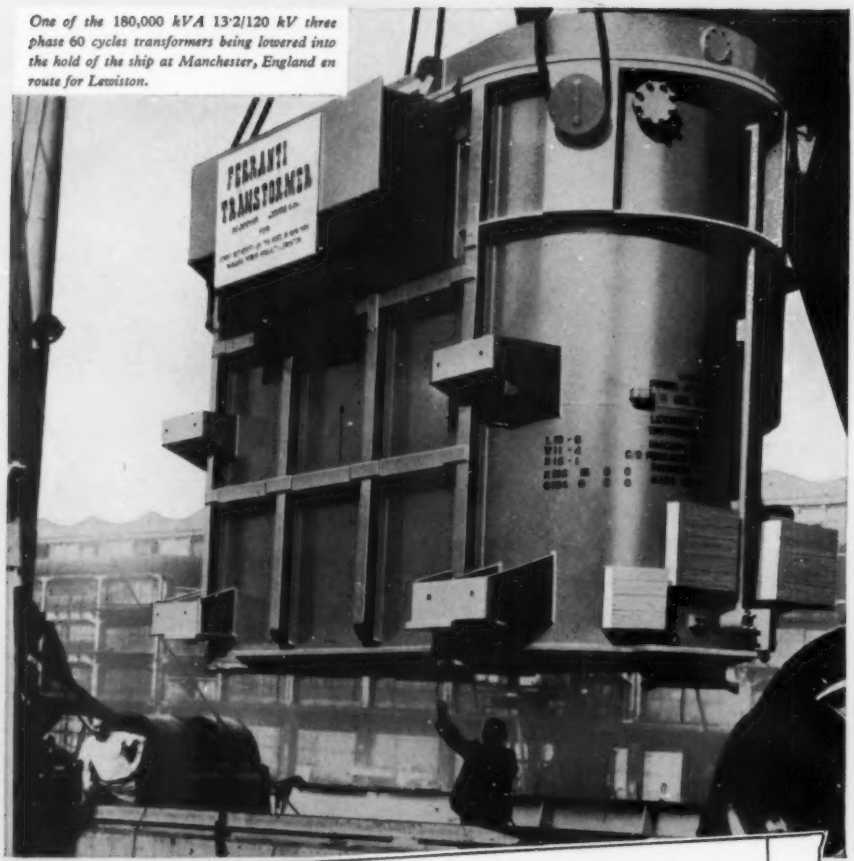
CWR Certain Cockburn-Rockwell Butterfly Valves including designs with spool liners, are manufactured to a common standard in Britain, United States, France, Holland & South America. Service and interchangeable spare parts for the CWR range are available in all these countries. Full information on request.

One of the 180,000 kVA 13·2/120 kV three phase 60 cycles transformers being lowered into the hold of the ship at Manchester, England en route for Lewiston.



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for
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Ferranti Ltd. are proud to have supplied six 180,000 kVA 13·2/240 kV and seven 13·2/120 kV three phase Generator Transformers for the Lewiston Power Plant of the Power Authority of the State of New York.

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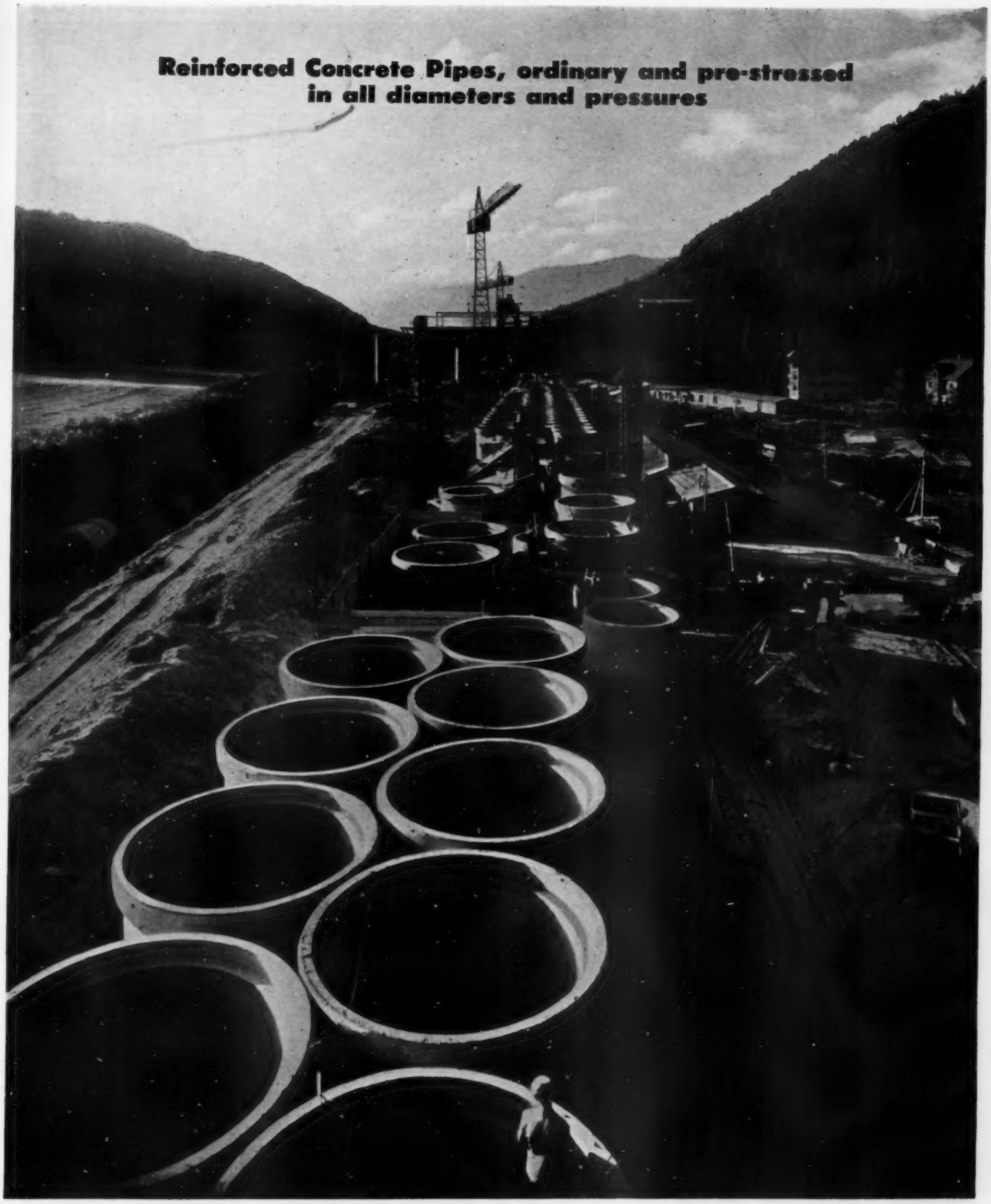

Pre-stressed reinforced concrete pipe, diameter 3.30m. pressure 9 atm. Brunico hydro-electric power station, Italy. Soc. Montecatini

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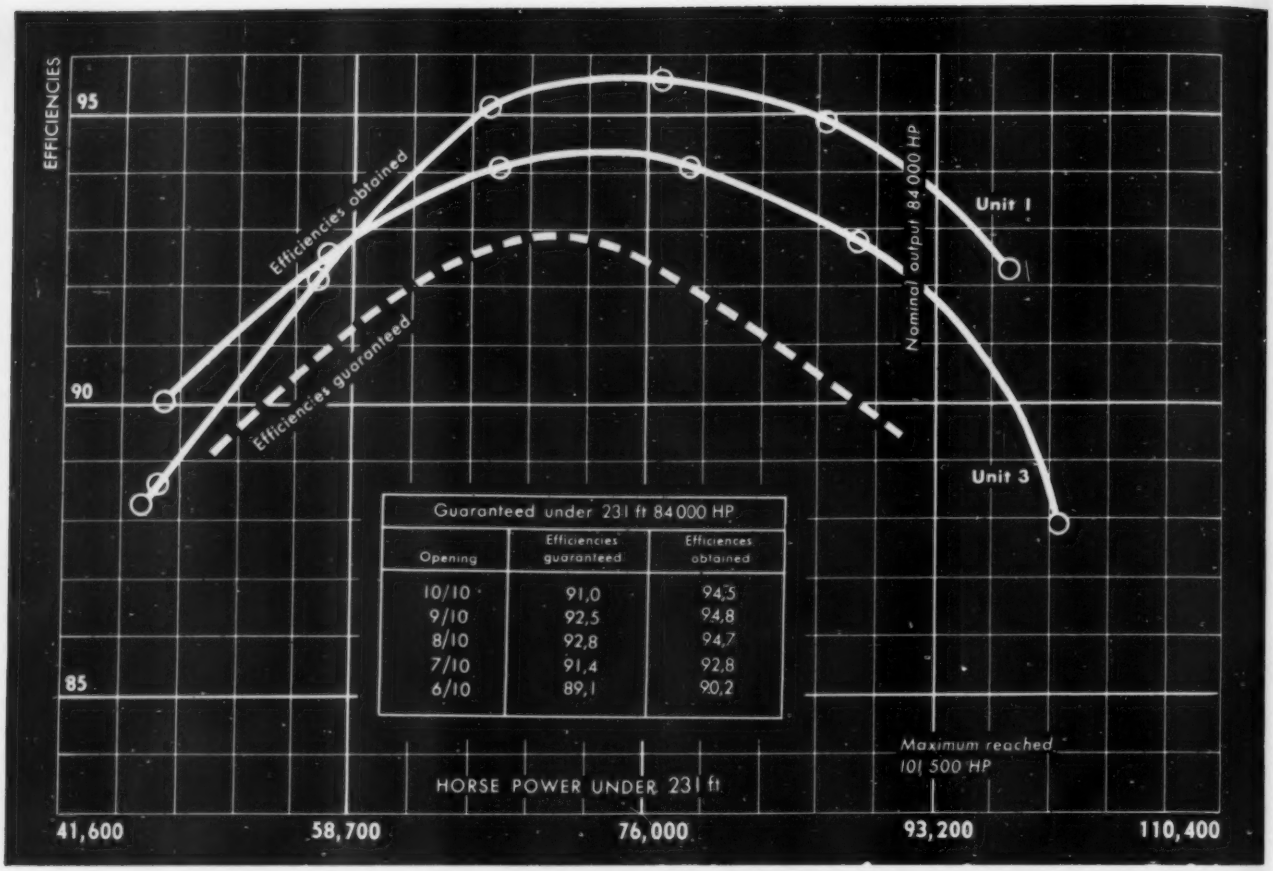

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PICOTE
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The sound qualities of Neyrpic turbines were again confirmed at Picote when the efficiencies of units 1 and 3 exceeded the guaranteed values

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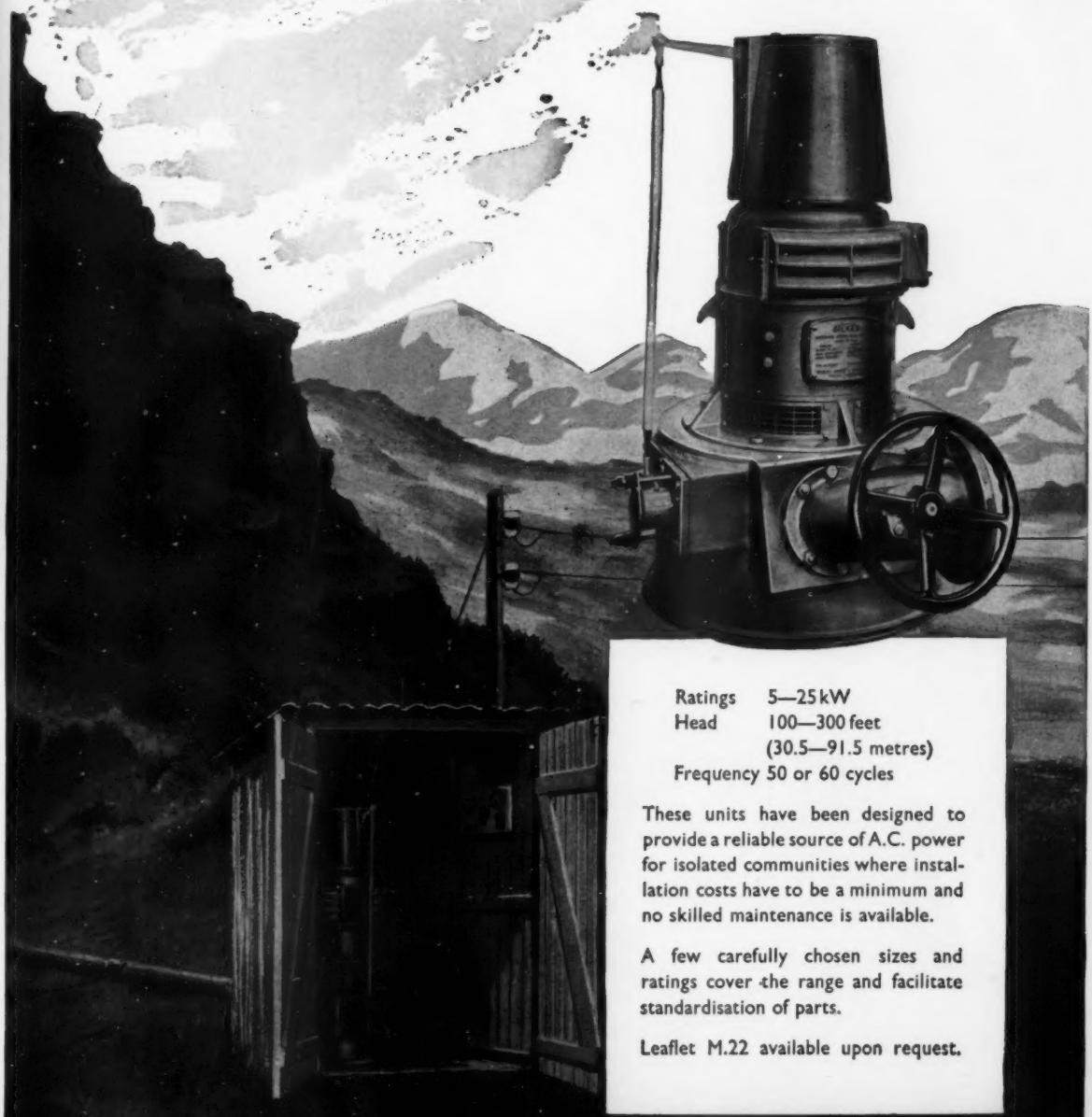

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Frequency	50 or 60 cycles

These units have been designed to provide a reliable source of A.C. power for isolated communities where installation costs have to be a minimum and no skilled maintenance is available.

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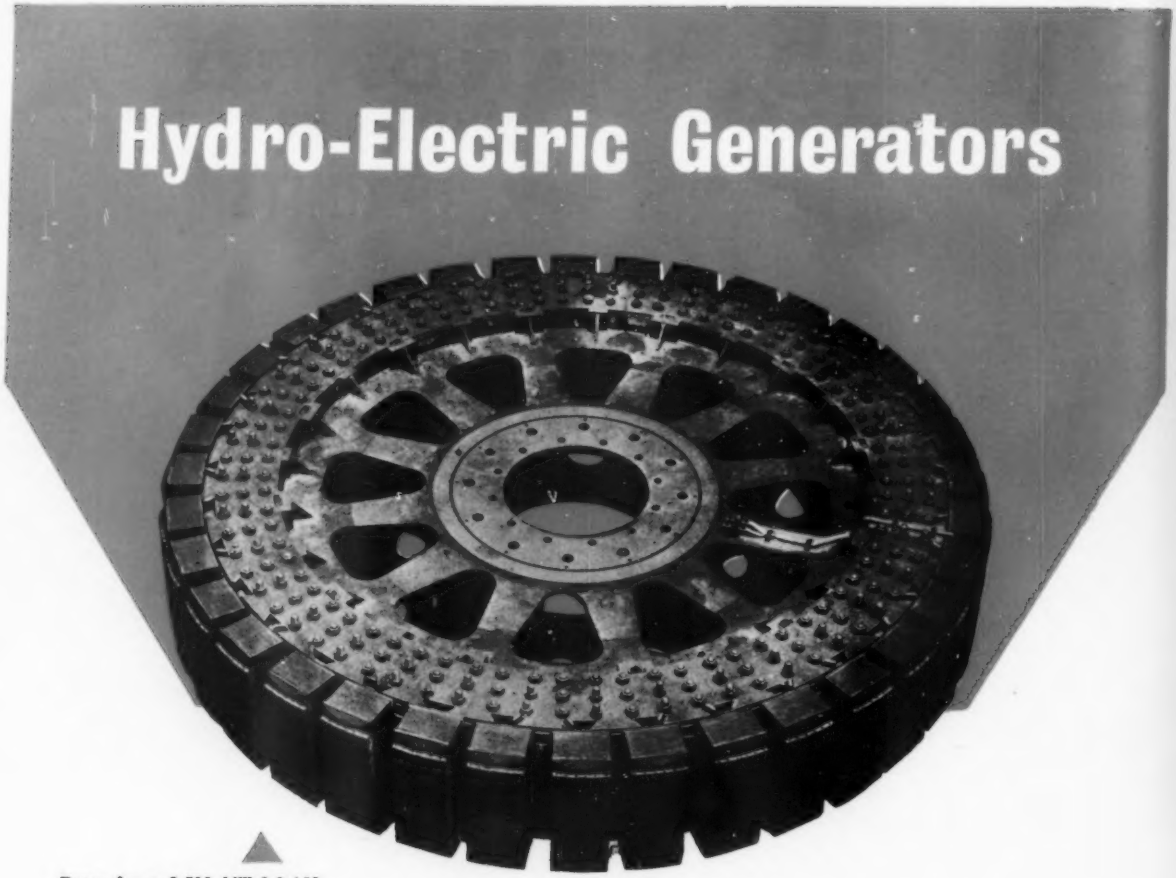
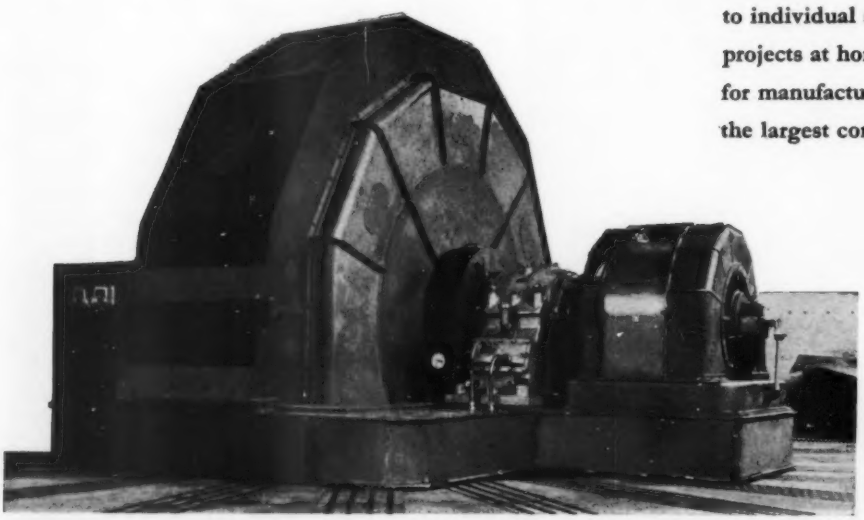
G121

Hydro-Electric Generators



Rotor for a 3,500 kW 3.3 kV 167 r.p.m. vertical generator for the Lairg power station of the North of Scotland Hydro-Electric Board.

Bruce Peebles build generators, large or small, horizontal or vertical, designed to individual specifications, for hydro-electric projects at home and abroad. Ample resources for manufacture and testing enable the largest contracts to be undertaken.



One of two 12 MW 11 kV 375 r.p.m. horizontal generators for the Shin power station of the North of Scotland Hydro-Electric Board erected for test.

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well on top of schedule

MOVEMENT of materials . . . . . that's the secret of meeting schedules on great projects. Getting the stuff in, putting the machinery on the job, - half the battle is won when movement is unhampered, when the equipment which handles the gear does so with unfailing reliability.

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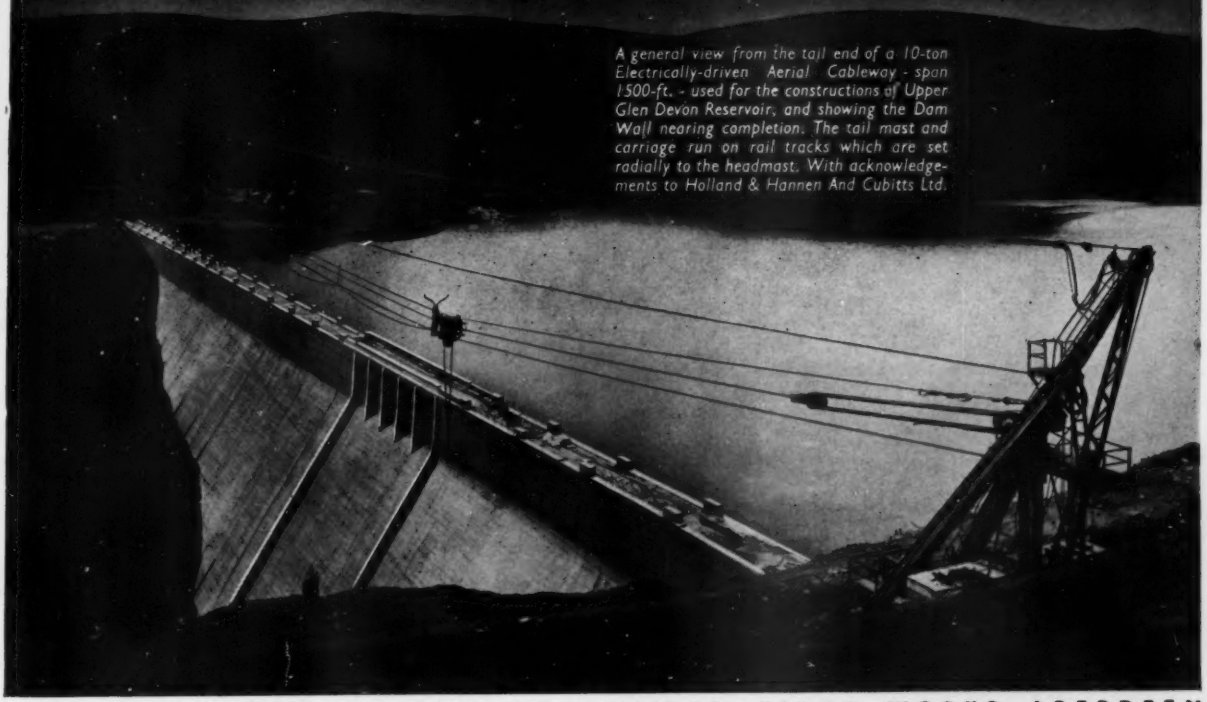

A general view from the tail end of a 10-ton Electrically-driven Aerial Cableway - span 1500-ft. - used for the constructions of Upper Glen Devon Reservoir, and showing the Dam Wall nearing completion. The tail mast and carriage run on rail tracks which are set radially to the headmast. With acknowledgements to Holland & Hannen And Cubitts Ltd.

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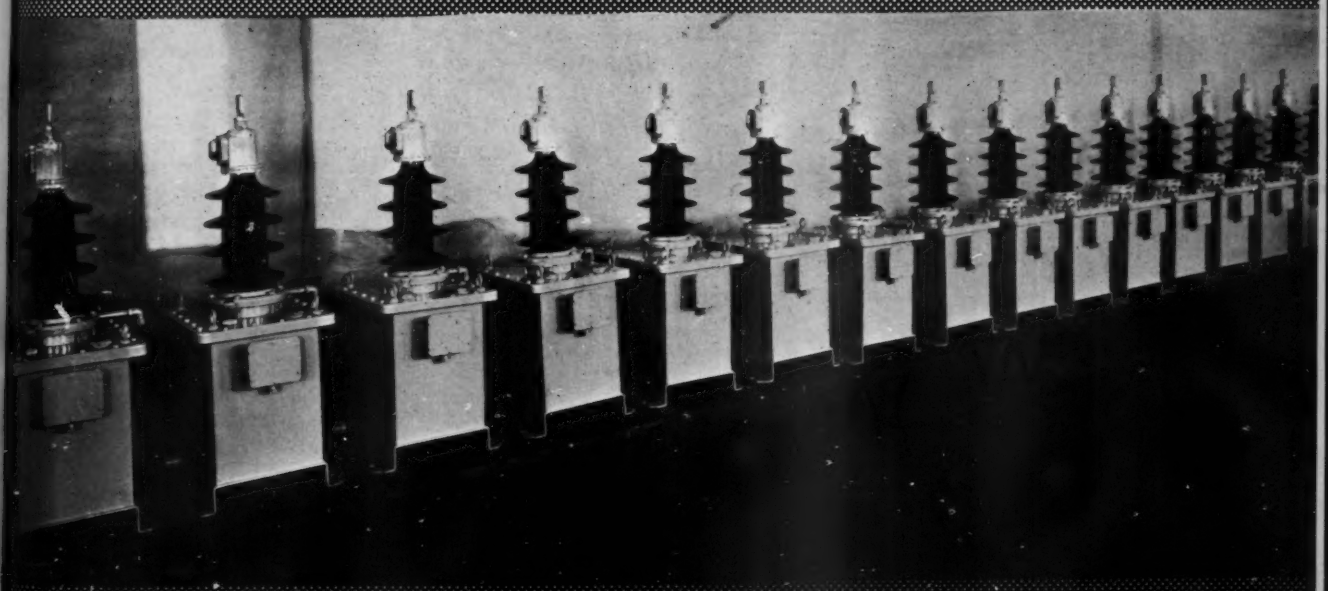
33 kV Potential Transformers for New Zealand Electricity Department



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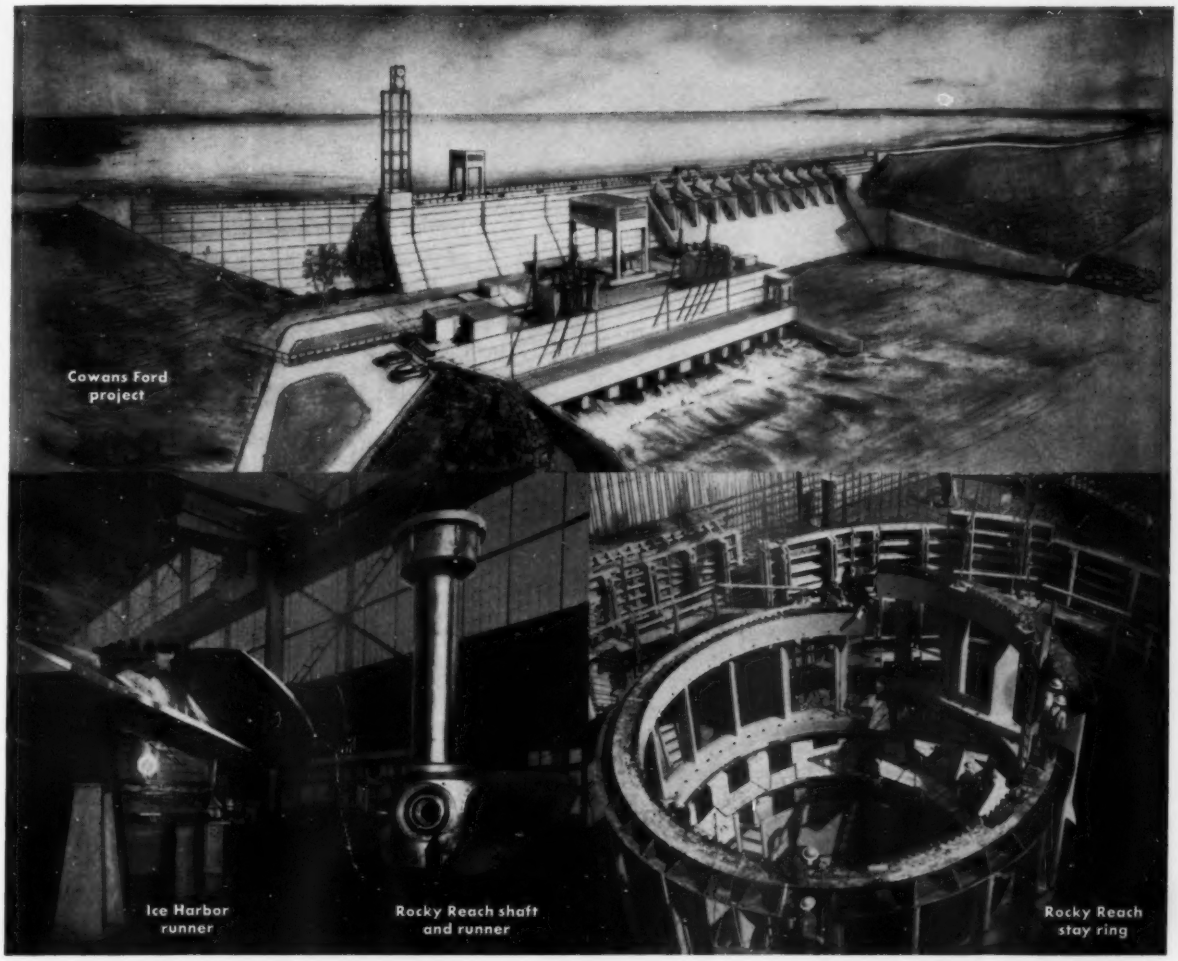

high-head Kaplans for peaking

—previous record-breakers also by Allis-Chalmers

Cowans Ford is the outstanding illustration of Kaplan turbines used to provide peaking capacity. Duke Power Company, with Chas. T. Main Inc. as consulting engineers, has selected A-C Kaplan turbines for operation at 115-ft. head. Each of the three units is guaranteed to provide a maximum of 153,000 hp. Actual operation will involve only a short time each day, meeting peaking requirements of the system which is predominantly steam. The resulting average load factor, *less than 5%*.

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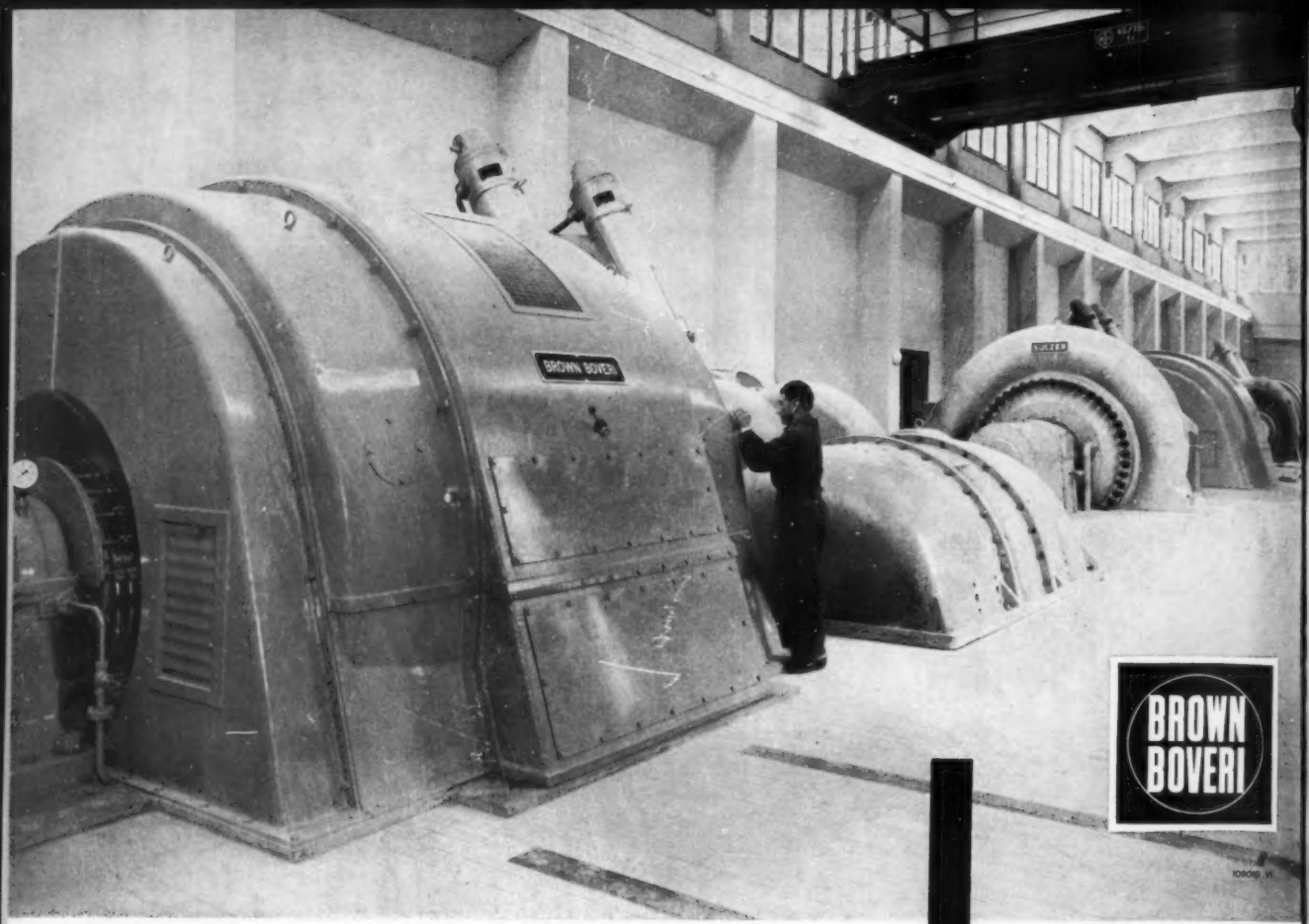
Rocky Reach—public utility district supplied with seven Kaplan turbines, each 140,000 hp (under 92-ft. head).

Ice Harbor — Corps of Engineers installation recently provided with three new A-C Kaplans for operation at 89-ft. head. These held the record with 143,000 hp per turbine until Cowans Ford.

In every facet of hydro generation, people count on record performance and experience from Allis-Chalmers. For assistance anytime, contact your local A-C representative or write to **Allis-Chalmers,** Hydraulic Division, York, Pa. A-1383



Machine room in the Motec power station of the Gougra Power Co., Switzerland with three 29.4-MVA generators for 750 rev/min 50 c/s

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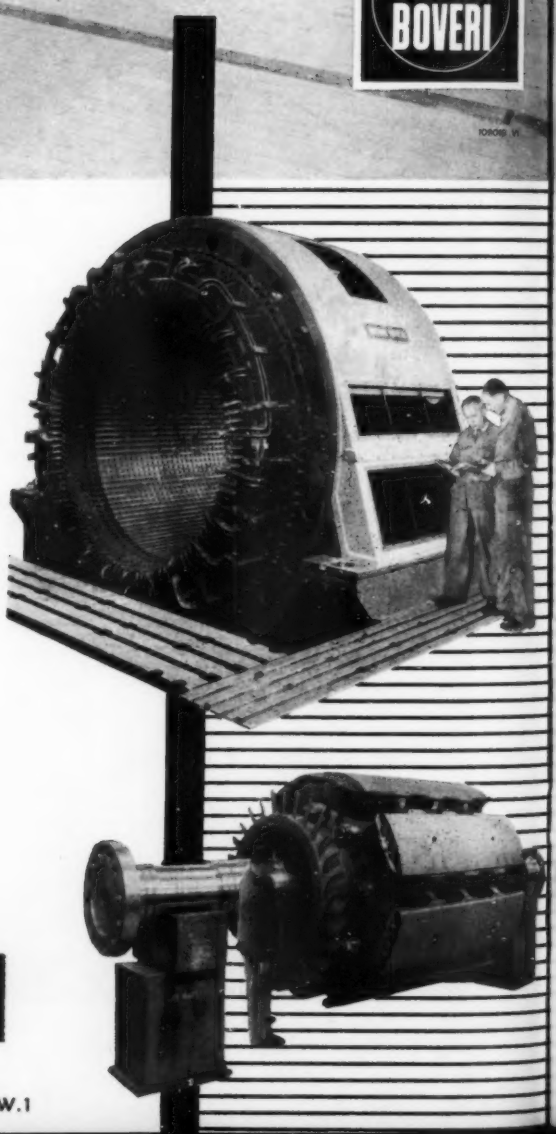

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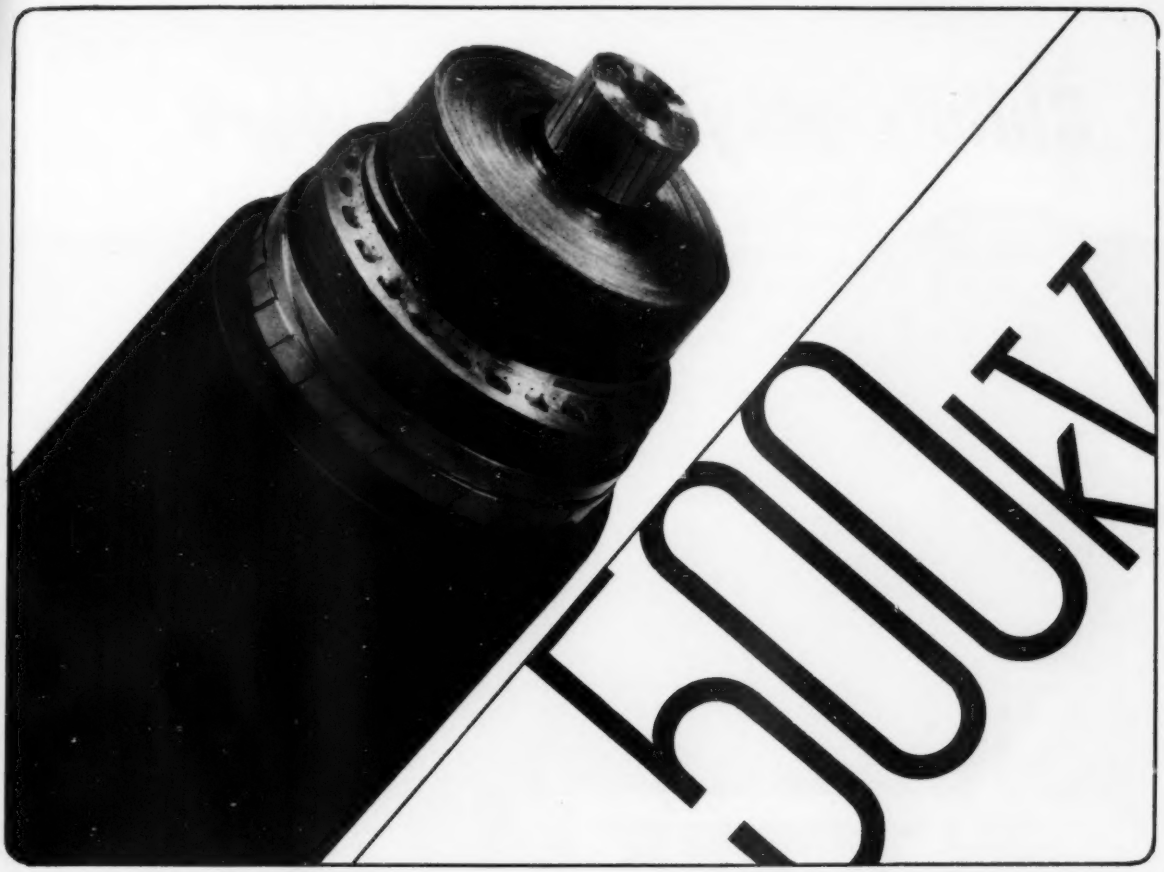

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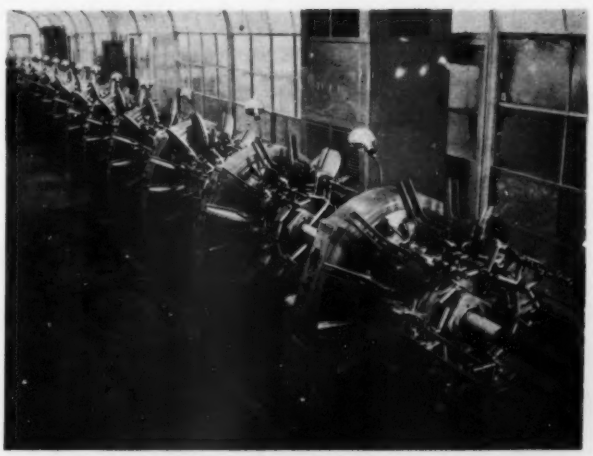
In keeping with the world trend toward a more extensive application of 400kV power transmission, Japan is now adopting the higher rating instead of the conventional 275kV as has been used in the past. This changeover has been reflected in intensified research activities at the ultra-high power transmission testing laboratory incorporated in our new plant at Hitaka. A recent technical tieup with the famous Swedish firm of Liljeholmens was yet another progressive step, and has already resulted in the development and successful testing of a 500kV class $1 \times 540 mm^2$ OF cable. Tomorrow's 400kV era will find us ready, as usual, at Hitachi.



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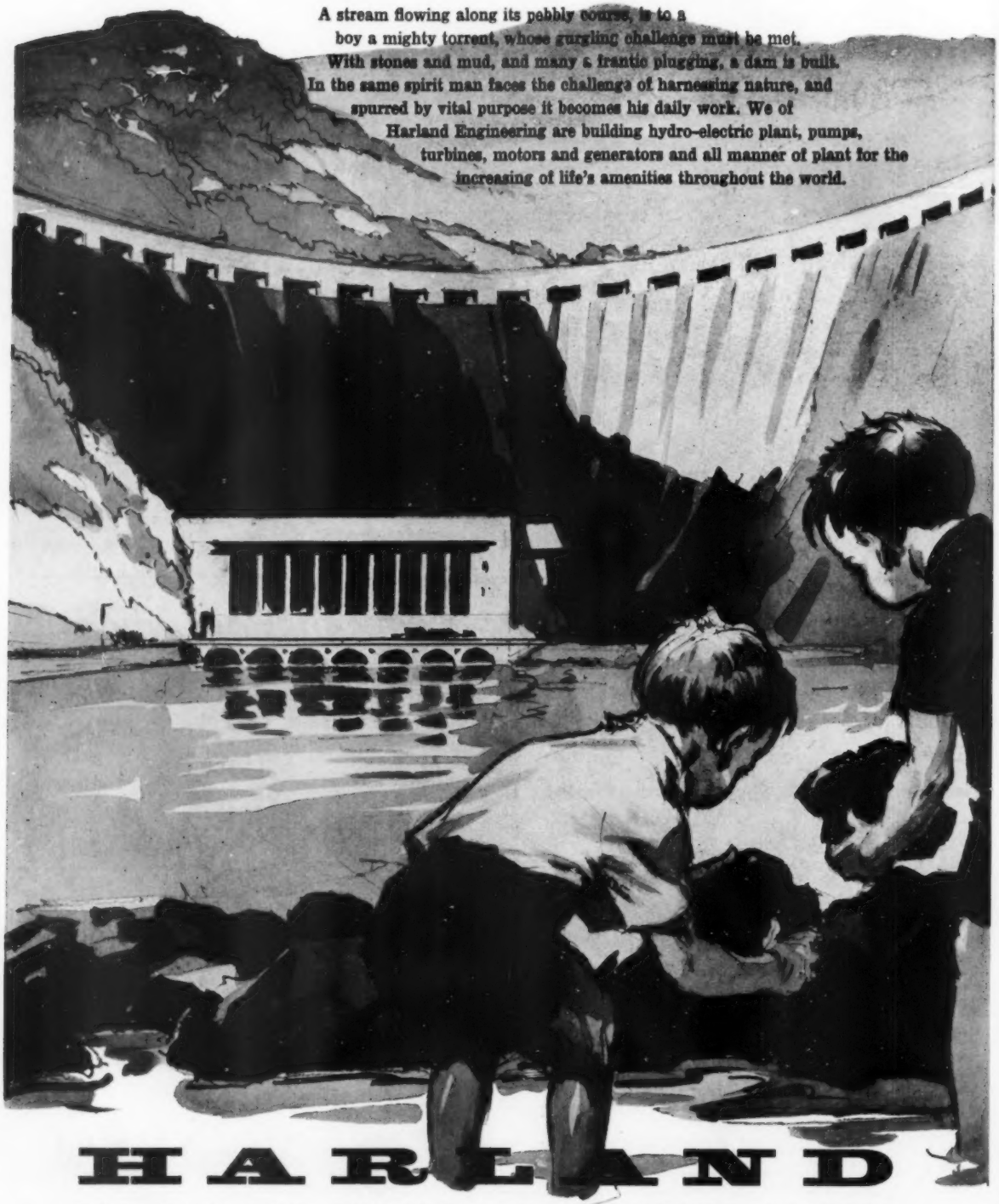

A stream flowing along its pebbly course, is to a boy a mighty torrent, whose gurgling challenge must be met. With stones and mud, and many a frantic plugging, a dam is built. In the same spirit man faces the challenge of harnessing nature, and spurred by vital purpose it becomes his daily work. We of Harland Engineering are building hydro-electric plant, pumps, turbines, motors and generators and all manner of plant for the increasing of life's amenities throughout the world.

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WATER POWER

A technical journal devoted to the study of all aspects of hydro-electric development

Some Light on Seepage Control

THE question of whether cut-offs beneath dams fully justify the time and expense usually devoted to them has long been a confused one. A clear-cut answer has yet to be formulated, and indeed may never be, but a considerable amount of air may have been cleared as a result of Professor Casagrande's lecture to the British National Society of Soil Mechanics and Foundation Engineering at the Institution of Civil Engineers on January 25, 1961. The lecture, which was entitled "Analysis and Control of Seepage for Dams," and was the first of the newly founded Rankine Lectures, was devoted to the problems of seepage and uplift in permeable dam foundations, and to the resolution of the problems by means of cut-offs, drainage wells, and impermeable upstream blankets.

The lecture was broadly divided into two parts, the first dealing with the problems associated with dams founded on pervious rock, and the second with dams on alluvial foundations. In both cases, however, the lecturer made the point that the efficacy of the grouted or sheet-piled cut-off might not be all that it appeared, and that much more thought ought to be devoted in the design stage to the necessity for such expensive works. Prof. Casagrande has formed the opinion, on the basis of his wide experience as a consultant, that, to many engineers, the provision of a cut-off has become almost a dogma, whereas proper drainage facilities have too often been relegated to a secondary place.

Prof. Casagrande dealt in detail with the nature of seepage, and the relief of uplift pressures beneath gravity dams by means of vertical drains. He expounded his theoretical approach to the design of these drains, and this in itself was a valuable contribution to the study of such systems. It was indicated that the diameter of the drains was of less importance than either their distance from the heel of the dam or the level of the drainage gallery, and, most important of all, it was shown that it was essential to carry the drains right down to the impervious stratum underlying the pervious foundation. The lecturer suggested that some existing vertical-drainage systems were only partially effective because this vital requirement had been overlooked. An account was given of the variations in uplift-pressure distribution produced by altering the distance of the line of the drains from the heel of the dam. If the actual reduction in the quantity of seepage was of importance, then the line could often be moved well back from the heel, although an increase in total uplift pressure would result.

Grouted cut-off curtains were discussed at length, from both the theoretical and the practical standpoints. Prof. Casagrande showed that a theoretical approach indicated that the efficiency of this type of cut-off might be expected not to exceed 30%, and might easily be as low as 10%. He produced a formidable body of evidence, mainly from observations made on U.S. dams, to show that the reductions in piezometric head, which were usually attributed to the grouted cut-offs, could possibly be due largely to the drains immediately behind the cut-off curtain. An extreme case was cited of an earthfill dam that had a grouted cut-off, which had been competently executed, but which remained subject to practically the maximum uplift pressure. No drains had been incorporated in this design.

Turning to the control of seepage in alluvial foundations, Prof. Casagrande discussed the merits of various types of cut-offs. He considered that concrete and steel-sheet piles were probably reasonably effective, particularly as they were driven at an early stage in the construction programme, and this allowed some time for a natural sealing process to take place before the reservoir was completely filled. In the case of steel piling, this sealing was assisted by corrosion of the clutches. He was of the opinion that the life of a steel-pile cut-off might be about 50 years, by which time the alluvium would, in any case, probably have become sealed with sediment.

The lecturer had a kind word to say for impervious upstream blankets because they tended to hasten the

natural process of sealing the permeable foundation material. This also applied to gravity dams on pervious rock, as Prof. Casagrande thought that there was often a case for the use of an earth seal at the heel of a dam. Impervious blankets were always worth considering at the design stage, and their use might be economically advantageous. It was evident from his remarks that the lecturer felt that any seepage-control system should be designed with the outlook in mind that the foundation might eventually become sealed as a result of natural causes.

It was unfortunate that Prof. Casagrande was obliged to abbreviate the latter part of his lecture, and, although he had sufficient time in which to cover his main points, many will await with considerable interest the publication of the lecture. It is believed that this will appear in the June issue of *Géotechnique*.

The lecturer was introduced by Prof. Skempton, who rightly said that Casagrande's part in the establishment of soil mechanics as a science was second only to that of Terzaghi, and in our opinion this excellent lecture has made a further valuable contribution to the science by restoring a measure of stability to an important branch of it.

Columbia River Treaty Signed

ONE of President Eisenhower's last acts before relinquishing his office was to join with Mr. Diefenbaker in signing the treaty between the United States and Canada for the joint development of the Columbia River. The provisions of this treaty, which is to run for 60 years, were outlined by Professor Marts in an article in our February issue. At the moment of writing, the treaty has yet to be ratified by the repre-spective governments, and the attitude of the Government of British Columbia towards the treaty is yet to be determined. Hitherto British Columbia is understood to have favoured the development of the Peace River, and the new treaty will raise important questions of policy. The British Columbia Government has already instructed the British Columbia Energy Board to investigate the comparative merits of the two schemes, and the Energy Board has now appointed two British firms of consulting engineers—Sir Alexander Gibb and Partners and Messrs. Merz and McLellan—to prepare a report on the comparative costs of power from the Columbia and from the Peace. Neither firm has been connected with either scheme, and the purpose of retaining them is obviously to obtain an independent opinion to guide British Columbia in deciding their policy.

Work to Start on Dominican Republic Tavera Dam

WORK on the Tavera-dam hydro-electric and irrigation project, which is believed to be the biggest single civil-engineering project yet undertaken in the Dominican Republic, was due to commence in January 1961. The dam, which will be 234 ft high, will impound a reservoir of 120 million m^3 capacity. A subsidiary dam, providing an additional storage of 1·5 million m^3, is also to be constructed.

The regulated flow will be utilised for the generation of an estimated annual output of 90 million kWh, and the supply of irrigation water to an area of 60,000 acres, the additional crops from which are expected to be worth some £15 million per annum.

The project, which will employ up to 4,000 workers, and which is expected to cost a total of $33,758,000, is to be carried out in three stages. The dam and associated works will first be built at an estimated cost of $16,758,000; secondly, the irrigation-canal system will be constructed at a cost of some $5 million, and, thirdly, the hydro-electric equipment, worth about $12 million, will be installed. At present, the bodies concerned in the scheme are the Public Works and Agriculture Department of the Government, and the Dominican contractors, Bonnet and Armenteros.

Rance Machines Ordered

ELECTRICITÉ DE FRANCE have given instructions to a group of French engineering firms to proceed with the design and construction of the bulb-type generating sets for the Rance tidal plant. The turbine manufacturers concerned are Ets. Neyrpic, Société des Forges et Ateliers du Creusot, Jeumont, and Alsthom-Charmilles. The electrical equipment will be supplied by Alsthom, Jeumont, and Schneider-Westinghouse.

Sweden's Power Consumption in 1960

SWEDEN'S power consumption in 1960 rose somewhat more than was estimated, according to a survey by the Stockholm financial weekly *Finanstidningen*. It was estimated that by the end of the year consumption would have risen by 2,500 million kWh, or 8·5%, as compared with 6·5% under previous forecasts. While the country's water-power reservoirs are at present filled to 80%, the comparatively dry autumn months in the northern part of Sweden, which accounts for about three-quarters of the country's hydro-electric power output, will make it necessary to resort to thermal power to a greater extent than was earlier believed necessary. The total output of thermal power during the winter season is estimated at about 1,000 million kWh, corresponding to some 300,000 tons of fuel-oil imports.

Weed Infestation on Lake Kariba

IT has been authoritatively stated that weed infestation by *Salvinia auriculata* on Lake Kariba is rapidly reaching emergency proportions, and that, as a result, several fishing stations on the lake have had to close down. The most likely solution seems to be the use of chemicals, which are expensive, and other possibilities include the introduction of a fungus or a virus, which might destroy the weed.

Investment Prospects in Norway

MR. TRYGVE LIE has inaugurated an "Invest in Norway" campaign, which is directed to foreign investors and is based on the potential wealth represented by Norway's water power and the industries it serves. For many years Norway has produced more electricity per head of population than any other country in the world; it has twice the water-power potential of any other country in Europe, and as yet this potential is only 25% developed.

According to the Norwegian Export Council, a special commission set up by the Government is now studying plans for the expansion of the aluminium industry which within the next ten years would make Norway the third biggest aluminium producer in the world.

Production rose from 120,000 tons in 1958 to 145,000 tons in 1959, and the projected output at the end of the ten-year period is about 800,000 tons. This industry would be supplied with power at 2 öre (one farthing, 0·3 US cent) per kWh at present price levels—some of the cheapest power in the world. Other important Norwegian industries supported by cheap power are wood-pulp and papermaking, chemicals and fertilisers, metalliferous and mineral mining, and fish and whale-oil processing.

The fact that the country's population is only 3·6 millions explains why Norwegian capital is insufficient to develop these resources to the full. A significant development within the last few years—after a period of natural reluctance—has been the conclusion of international agreements for the export of power. A joint Norwegian/Russian development is in progress on the Pasvik River, and two stations are being built on the River Nea, with the aid of Swedish capital, to export power to Sweden for a period of 20 years.

American Niagara Power Plant Commissioned

THE State of New York Power Authority's power plant at Lewiston on the Niagara, the civil-engineering works of which were described in our August 1960 issue, was inaugurated on February 10, when Governor Nelson A. Rockefeller and Robert Moses, Chairman of the Power Authority, switched the first power on to the lines. The station has been named the Robert Moses Niagara Power Plant. All its 13 generating units will be on the line about a year hence, and will consist of Baldwin-Lima-Hamilton and Newport News 200,000-h.p. turbines driving Westinghouse 150-MW generators and supplying 115-kV and 230-kV systems through Ferranti 180-MVA transformers.

The associated Tuscarora pumped-storage plant will commence operation next year, and the entire project will be completed in 1963.

Inauguration of the Warsak Project

THE multipurpose Warsak project (WATER POWER, November and December 1960) was handed over to the President of Pakistan in late January. The construction has taken some five years to complete, and the total cost is about Rs. 350,000,000.

For the time being, the Warsak station is generating some 50 MW for commercial and domestic use. The existing installed capacity will be fully utilised later in the year when a new transmission system is completed. Later it is planned to add two more units, which will raise the total installed capacity to 240 MW.

Second Thoughts on Nkula

IT is reported that doubts have now arisen about the advisability of proceeding with the Nkula Falls scheme in Nyasaland at present. Although a loan from the Colonial Development Corporation is being sought by the Federal Government of Rhodesia and Nyasaland, it has been stated that an early start on the scheme depends on the load forecast for the protectorate. In September, 1960, the Ministry of Power announced that a start would be made within a year, but development in Nyasaland has apparently slowed down since then, and the Ministry is unlikely to proceed unless it is satisfied that the project will be economical. It has also been stated that, in order to develop Nkula, the Federal Power Board would have to pledge assets at Kariba, which it could not do unless a financial return from Nkula was assured.

Taum Sauk Machinery

FURTHER to our article on the Taum Sauk pumped-storage plant in our February issue, we are now able to report that the order for the reversible pump-turbines has been placed with Allis-Chalmers Manufacturing Company, and for the generator-motors with the General Electric Company. The pump-turbines will each have a capacity exceeding 300,000 h.p. at 800 ft. head, and the electrical machines will be rated at 225–175 MW at heads from 858–745 ft. When completed, these machines will probably be the most powerful in the western world. The power-house structure and the hydraulic works are being executed by Fruin-Colnon Contracting Company, of St. Louis, and the Utah Construction and Mining Company, of San Francisco. It is expected that Taum Sauk will be built for about $115 per kW, excluding transmission.

New Zealand Submarine-Cable Link

IT has been announced by the New Zealand Minister of Electricity that the experimental power cable which was laid on the bottom of Cook Strait, which separates the North and South Islands of New Zealand, has been found to be in excellent condition. It was recovered in February 1960 after being submerged for a period of two years, and has subsequently been closely examined and tested by the manufacturer (BICC) in England.

The results of water-velocity measurements made in the strait were also satisfactory. These showed that, although strong currents exist, their velocity is substantially reduced near the sea bed.

Following the conclusion of these tests, preliminary advice has been given to the Minister by the new power-planning committee that the Cook Strait cable is to be recommended. Its installation is considered essential to the industrial and residential development of both islands. The final report of the committee is to await the discussion of details with the consultants.

Preliminary Works Begin at Manapouri

FOLLOWING the information given in the February 1961 issue of WATER POWER, we now learn that detailed site investigations and surveys have already commenced at Manapouri in New Zealand. In addition to topographical surveys and ground control for aerial surveys, an extensive programme of exploratory drilling and pilot-shaft sinking is to be carried out. A shaft will probably be sunk in the western arm of Lake Manapouri, and this will be carried to a depth of some 600 ft, at which level the power station is likely to be sited. It is hoped to have all the site investigations completed and designs ready for tender by the end of 1961.

The development company, known as Comalco Aluminium Products Pty. Ltd., intends now to proceed with a first-stage development that will produce 280 MW for the Invercargill smelter, and it is estimated that the ultimate capacity will be 600 MW. The most likely initial development will entail only a

low barrage on the Waiau River, which is the natural outlet from Lake Manapouri. The water will be led to the underground power station through the proposed shaft, and discharged through a 6½-mile long tailrace tunnel to Doubtful Sound.

It is currently estimated that the total cost of the initial development will be £A130 million, including some £A40 million for the alumina plant and facilities at Weipa, Australia, and about £A9 million for the Bell Bay aluminium plant in Tasmania.

Third Set for New Brunswick Station

WORK on the installation of the third unit at the Beechwood hydro station of the New Brunswick Electric Power Commission has been scheduled to commence in early 1961 and completion is anticipated a year later. The total cost of the new unit including the turbine, generator, unit transformer and all auxiliary equipment will amount to approximately $2,800,000. Provision for the installation of the third unit was made in the power house at the time of construction as part of the Commission's long-term plan for power production.

The turbine, which is being built by the Canadian Allis-Chalmers Company of Lachine, Quebec, will have a rated capacity of 55,500 h.p. The contract for the 45-MW generator has been let to the Canadian Westinghouse Co. Ltd. The new unit will be slightly larger than the two sets that when on line in the late autumn of 1957.

Revised Scheme for Ceylonese River

THE original proposal for the development of the Seven Virgins site in the Maskeliya valley in central Ceylon has now been abandoned following an adverse report on the foundation conditions by an International Co-operation Administration study group. The scheme proposed a development of some 160-MW capacity.

It is now proposed that the valley should be developed by the construction of three separate power stations at Morahengama, Theberton and Laksapana. To supply these stations, it is planned to construct a dam and reservoir near Mousakelle, a diversion dam at Laksapana, and a third dam at Theberton. It is estimated that the revised scheme will cost about Rs. 275,000,000, and work is to commence on the project as soon as funds are available.

Canadian Progress in 1960

THE Water Resources Branch of the Department of Northern Affairs and National Resources, Ottawa, has issued its annual review of current water-power development throughout Canada, and of other construction pertaining to electric power supply and distribution. During the year under review, a net total of 1,741,820 h.p. of new hydro-electric capacity was added in Canada after allowing for 4,680 h.p. which was dismantled or destroyed. Quebec's total of 1,176,500 h.p. of new capacity installed during 1960 surpassed the combined total of the other provinces. Manitoba and British Columbia each added slightly more than 200,000 h.p. and ranked second and third, respectively, in the amount of new hydro-electric capacity brought into operation during the year. The total installed capacity of water-power plants in Canada is now listed at 26,372,444 h.p. New capacity to be added in Canada during 1961 is limited to about 243,000 h.p., but construction is proceeding at other developments where 4,500,000 h.p. is expected to be installed and where provision will be made for installation of an additional 1,000,000 h.p. in later years.

The greatest increase in thermal-electric capacity during the year occurred in Ontario, where the development of the principal hydro-electric sites adjacent to present load centres has been largely completed. There is a growing tendency towards increased thermal-electric generation in Canada, which reflects not only a stage of hydraulic development wherein available undeveloped sites are becoming more remote from established demand areas, but also the benefits of resource conservation that may be derived through the operation of an integrated power system supplied by both hydro-electric and thermal-electric plants.

Swedish Turbines for Norway

TWO water turbines have been ordered from NOHAB, Trollhättan, by A/S Kykkelsrud, Oslo, to be installed in the Kykkelsrud power plant, Western Norway. In order to get the best hydraulic qualities of the plant, tests will be made by the Technical University in Trondheim, Norway, regarding the placing of the plant and the dam in the river. The turbines, which are to be of the Kaplan type, will be placed in concrete spirals. They will develop about 55,000 kW at a head of 25 m. The Kykkelsrud turbines will be in operation in the middle of 1963, and they will cost about Sw. Kr. 7,500,000 (£517,000).

McKay Creek Power Station Completed

SINCE May 1960, the State Electricity Commission of Victoria, Australia, has put 96 MW of generating capacity into service at its new McKay Creek hydro-electric power station, which is situated on the slopes of Mount McKay in the Australian Alps, and which forms part of the Kiewa hydro-electric project. The first of six 16-MW generator sets, supplied by Associated Electrical Industries Limited, Rugby, England, was commissioned at the beginning of May 1960, and the last has recently been put on load. Three of the four power stations planned for the Kiewa development are now in operation, and they have a combined output of nearly 184 MW.

The McKay Creek power station, which is situated underground, is supplied by the Rocky Valley reservoir, which is about a mile above sea level and is stated to be Australia's highest reservoir. The station is controlled from Mount Beauty, a township 17 miles away. Water for the Boving turbines is conveyed through nearly three miles of tunnel, thence by surface pipeline, and finally through a 630-ft steel-lined pressure tunnel. The turbines work under a head of more than 1,700 ft. After passing through the McKay Creek turbines, the water is discharged into the East Kiewa River, whence it supplies the turbines of No. 3 power station. The water is utilised a third time by being diverted underneath the mountain to the No. 4 power station, on the West Kiewa River. Power is supplied from these stations to Melbourne, 153 miles away, at 220 kV.

Fig. 1. A view of the completed Cancano dam

The Valtellina Development

A development using the headwaters of the River Adda in the Italian Alps has been replanned for an ultimate output of 1,500 GWh per annum. This first article describes the new Cancano dam, which has been designed for construction in two stages

PART ONE

As long ago as 1906 the Azienda Elettrica Municipale of Milan commenced the construction of a power station at Grosotto on the River Adda in Valtellina, and this station, of a capacity of 36·2 MW, entered into service in 1910. This was followed by Roasco, of 20-MW capacity, in 1922, and in 1928 the first reservoir was impounded at Cancano with a capacity of 24 million m^3 at a retention level of 1,857 m to serve a 45-MW station at Fraele. Succeeding stations to be built were Stazzona (35 MW, 1938), Lovero (44 MW, 1948), and in 1950 a reservoir was constructed of 64 million m^3 capacity with a retention level of 1,949 m at San Giacomo, immediately upstream of Cancano reservoir, together with a small power station of the same name. A side-stream tunnel was also constructed to divert the headwaters of the Braulio and Forcola mountain streams into San Giacomo reservoir. At this stage the system was as shown in the map, Fig. 3, and produced an average of 750 GWh per annum.

During the war Azienda Elettrica Municipale obtained the concession to develop the hydraulic resources of the Valfurva, and it became possible to replan the entire development as an integrated system. The changes will be evident on comparing Fig. 4, which is a map of the redeveloped scheme, with Fig. 3. A new arch dam has been built some way below the existing dam at Cancano to increase the live storage of this reservoir ultimately to 240 million m^3, at a level of 1,936 m, and this water is being used to supply an underground station at Premadio, which will have an ultimate capacity of 375 MW at a head of 708 m. The discharge from Premadio feeds a second station at Grosio, which will ultimately be of

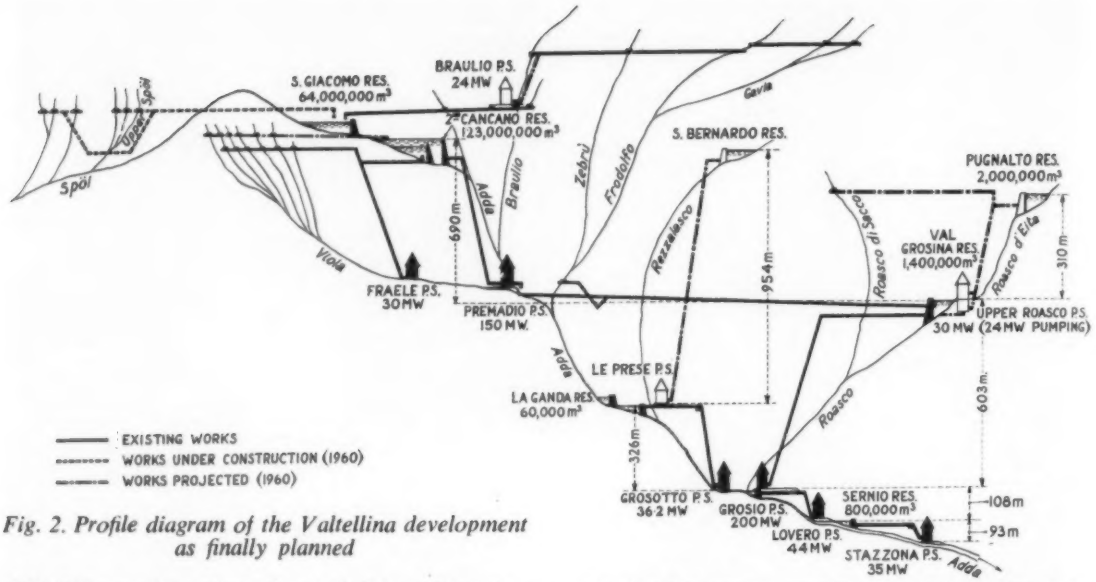

Fig. 2. Profile diagram of the Valtellina development as finally planned

400-MW capacity under a head of 600 m. Diversion tunnels have been built or will be constructed to direct all waters above a level of 2,000 m into the San Giacomo and Cancano reservoirs. On the left of the Adda the tunnel carrying the headwaters of the streams in the Valfurva has already been built and will later connect a 1 million m³ reservoir at Forni with a future station at Braulio. On the right of the Adda a side tunnel has been built to collect the headwaters of the Viola Bormina and an intake system is projected to take in the headwaters of the Spöl, on which a reservoir of 180 million m³ will be constructed. A 2 million m³ reservoir is also contemplated on the Roasco to supply an Upper Roasco station, after which the old Roasco and the Fraele stations will be dismantled.

When completed, the development will be able to generate about 1,500 GWh per annum, mainly from regulated waters so that it can be used to firm up base-load power from thermal and atomic stations.

To match the rate of installation with the growth of load and to keep financial commitments within control, construction has been phased in a series of four-year periods. Between 1952 and 1956 the new Cancano dam was built to an initial height of 136 m to impound 123 million m³, at a level of 1,900 m, and Premadio station was constructed with an initial installation of two 75-MW sets. Between 1956 and 1959 Grosio station was built with an installation of 200 MW—half its ultimate capacity. Future periods will see the heightening of Cancano dam to 172 m to give the full storage (when the combined head storage of the San Giacomo and Cancano reservoirs will amount to 304 million m³), the installation of the remaining generating sets in Premadio and Grosio stations, and the completion of the other reservoirs, diversion tunnels and stations mentioned above and indicated in Figs. 2 and 4.

The new Cancano reservoir is sited on the conjunction of four valleys—the Viola, Fraele, Braulio and Valfurva—and the decision to provide such an ample head storage was determined by the hydrological characteristics of the region. The catchment is sheltered from the moist southerly winds, with the result that the annual precipitation is less than 900 mm except in elevated and exposed localities, where it rises above 1,000 mm. On the other hand, the snow-covered regions have a precipitation equivalent to 1,500–1,600 mm and in consequence the flow is

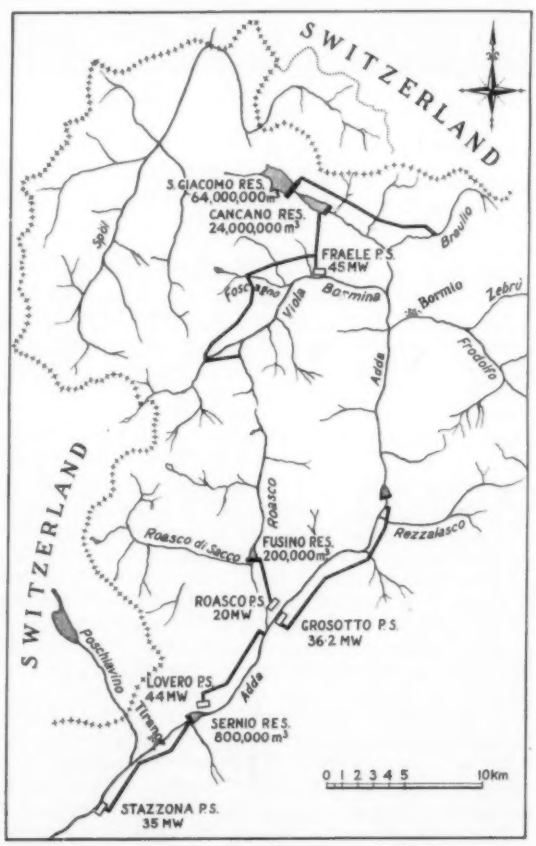

Fig. 3. The Valtellina development in 1954

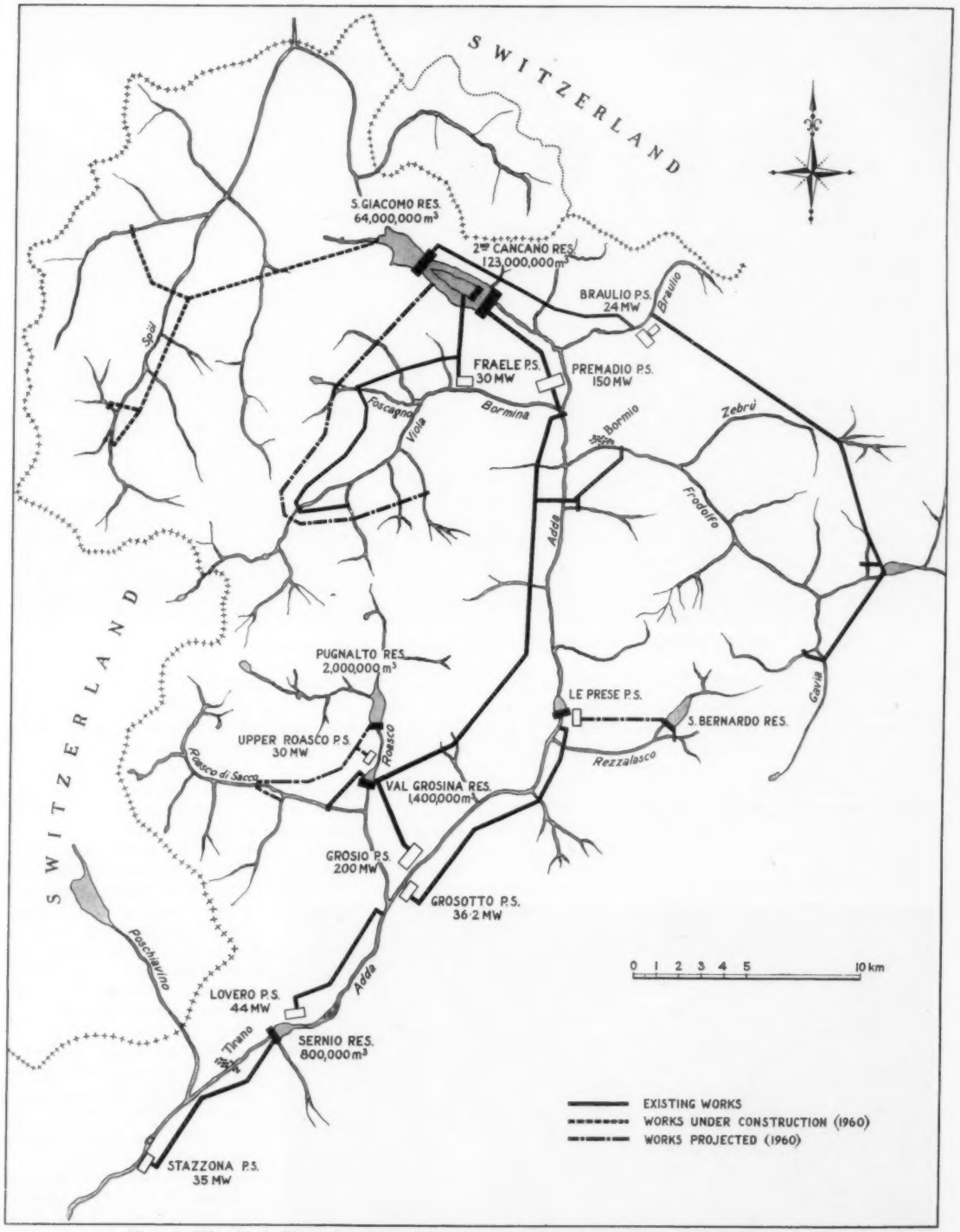

Fig. 4. The Valtellina development as now planned, showing progress to date

markedly seasonal, the main flush occurring in the late spring. The maximum demand from the industrial districts of Milan, however, occurs in the winter, and the Cancano reservoir has therefore been designed to provide sufficient seasonal storage to cover this condition.

The catchment area draining directly into the Cancano reservoir is 36 km². The Valfurva-Braulio diversion adds 149 km² and the Viola diversion 79 km², making a total of 264 km². The future diversion of the Spöl will add a further 105 km², making a total of 369 km².

TABLE I.—QUANTITIES FOR VARIOUS TYPES OF DAM

Type of dam	Height	Excavation		Cement	Concrete	
	m	m³	%	kg/m³	m³	%
Mass gravity	150·0	90,100	100·0	210	719,400	100·0
Hollow gravity	150·0	110,000	122·2	230	618,000	85·9
Arched gravity	150·0	158,400	176·0	240	605,500	84·2
Cupola	133·5	151,900	168·8	270	466,400	64·8

Diversion Tunnels

Disregarding the future proposals for the Spöl and the Roasco, the immediate programme calls for the construction or completion of two diversion tunnels —the Valfurva entering from the left bank and the Viola entering from the right.

The Valfurva tunnel was constructed in three successive stages, commencing with the one farthest downstream—that between the Braulio and San Giacomo reservoir. This section, which is 10 km long, was originally driven in 1943 to a reduced section, but was enlarged to a section of 9·4 m² to carry 19 m³/sec in free flow. The tunnel is seen in Fig. 5.

The second section, between the Braulio and the Forni valley, where a compensating reservoir is to be built, was commenced in 1951. It is 16 km long and ranges from 4·8 to 9 m² in section, to take a maximum flow of 19 m³/sec. The Forni intake has been designed to accept the flashy flows that occur particularly in the summer, and is provided with a Dufour-type desilting basin with supplementary discharges and with a scour conduit, which extends the whole length of the future reservoir site. Five side intakes join this section of the tunnel at various points. Eventually this section will supply a power station at Braulio at a head of 180 m, and the tunnel has accordingly been built to withstand a head of 40 m at its downstream end.

TABLE II.—COST COMPARISON BETWEEN VARIOUS TYPES OF DAM

Type of dam	Dam only	Dam and discharge works
	%	%
Mass gravity	100·0	100·0
Hollow gravity	102·1	103·2
Arched gravity	99·2	103·8
Cupola	81·9	90·8

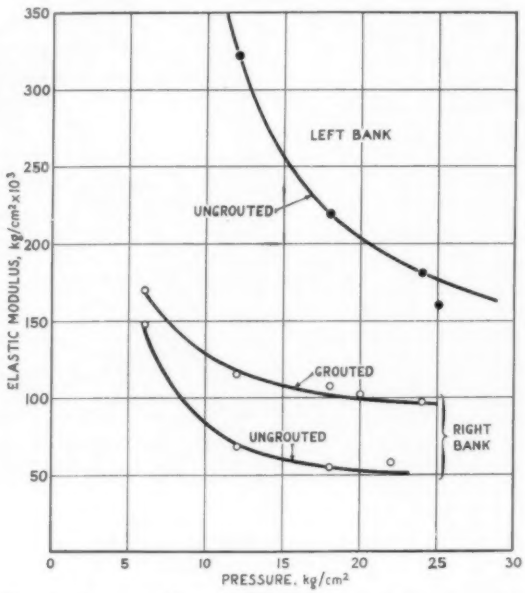

Fig. 6. Tests on the elasticity of the abutment rocks in Cancano gorge

Fig. 5. A view inside the Valfurva tunnel

The final section, completed in 1958, is that between the Forni and the Gavia valleys. It is 7·7 km long and has a section of 2·7 m², designed to carry a free flow of 8 m³/sec.

The total length of tunnel in the left-bank diversion works is thus upwards of 33 km.

A new Viola diversion tunnel is to be constructed to replace the earlier diversion (shown in Fig. 3) supplying the Fraele station from an intake level of 1,857 m. This earlier tunnel was designed for a maximum flow of only 7 m³/sec and was not large enough to take proper advantage of the spring melt. The new tunnel will be 20 km long and will carry a free flow

Fig. 7. Cancano dam from the upstream side, showing the intakes

of 11 m³/sec. For the present it will carry the entire flow to Cancano reservoir as long as the storage level is below 1,857 m; above this level the flow will spill to supply Fraele station. Eventually, of course, Fraele station will be shut down.

The New Cancano Dam

From the figures given above it will be noted that Cancano reservoir will ultimately have a capacity of ten times its original storage. To impound this greatly increased reserve it was decided to build a new and higher dam in the gorge of the Adda some way downstream of the old dam, which would be opened and submerged. The backwater would be allowed to rise part way up the downstream face of the San Giacomo dam but would not reach the crest.

A suitable location for the new dam was presented by a deep vee-shaped gorge about 1 km long, cut in a generally sound limestone dipping upstream and towards the left bank. Several possible sites were available along the gorge, depending on the type of dam, and although the conditions obviously called for a concrete dam it was an open question what would be the best type of structure. Comparative studies were therefore made of four possible designs on sites best suited for the structures concerned. These comprised a cupola arch dam on a site farthest upstream, studied by Prof. G. Oberti; an arched gravity dam farther downstream, studied by Prof. A. Berio, and, still farther downstream, a hollow gravity dam and a mass-concrete gravity dam, studied by Prof. F. Contessini. At the time that these studies were made the amount of storage to be held had not been decided, so a retention level of 1,910 m was tentatively adopted.

A comparison of the quantities involved in the construction of these alternative designs, summarised in Table I, showed that the mass-concrete dam called for the least excavation but the most concrete, whereas the cupola dam was at the other end of the scale on both counts except that the arched gravity dam called for an even larger amount of excavation. Incidentally, the lower height of the cupola dam is explained by the fact that the site selected was the farthest upstream.

The comparison was made much clearer by reducing it to a basis of costs, and further light was thrown on the question by taking into account the cost of the

TABLE III.—COMPARISON BETWEEN TYPES OF POZZOLAN CEMENT

Type of cement	Proportion of cement by Bolomey formula		Minimum proportion of cement 250 kg/m³	
	Proportion of cement	Heat of hydration	Proportion of cement	Heat of hydration
	kg/m³	cal/m³	kg/m³	cal/m³
Marine	198	11,642	250	14,700
Low-heat	275	15,290	275	15,290
Normal	223	15,276	250	17,125
Ferro-pozzolan	268	14,258	268	14,258

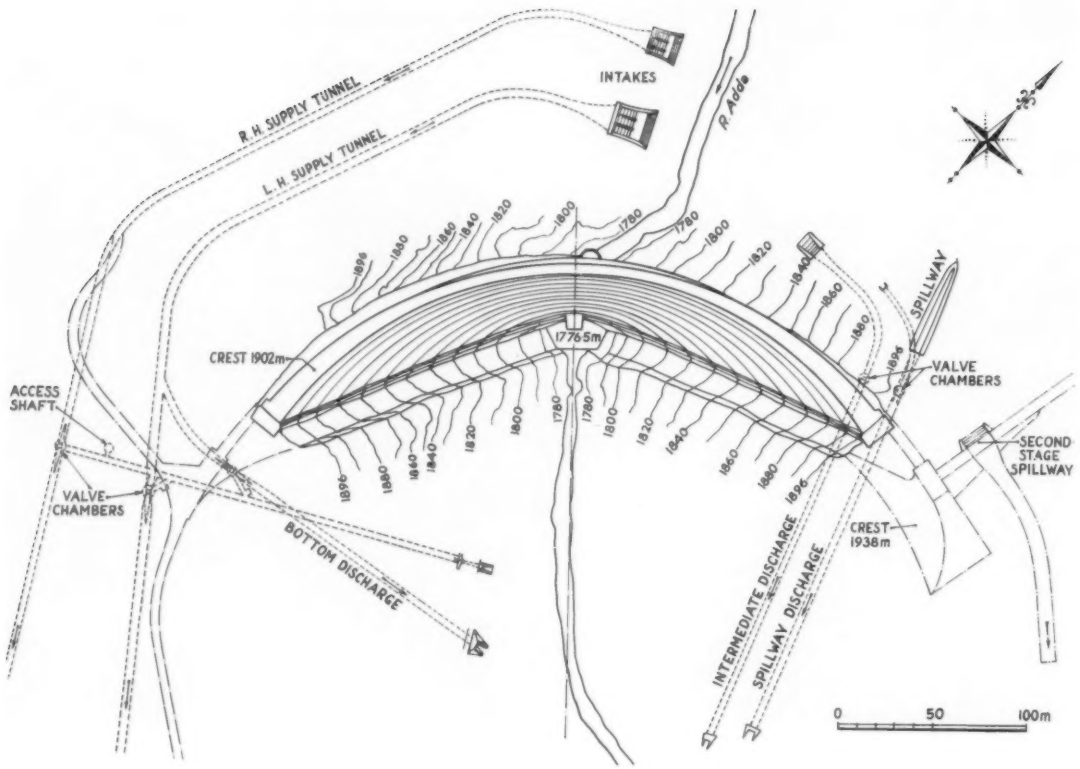

Fig. 8. Plan of Cancano dam. The projected second stage is shown chain dotted

associated discharge works. These figures, shown as percentages in Table II, showed the cupola dam to be unquestionably the most economic.

In the meantime the ultimate storage in Cancano reservoir had been fixed at 240 million m³, corresponding to a retention level of 1,936 m and a dam height of about 170 m, and the desirability of suiting the pace of the development to the growth of electrical demand and to the problem of financing gave rise to the proposal to build the dam in two stages, the first stage to impound 114 million m³, with a dam height of 132 m. This virtually precluded the use of a cupola dam because of the constructional difficulty of thickening and heightening a dam with an overhanging arch, and it was decided to resort to an arched gravity type of dam, but with a much thinner section than had been adopted in the original comparison, considerable reliance being placed on arch action to secure the strength of the dam without exceeding reasonable stresses. For this solution the site previously selected for the arched gravity alternative was obviously the most generally suitable, but before making a final decision two alternative locations on this site were studied. These two locations shared a common buttress on the left-hand bank but the buttresses on the right-hand bank were at different points, so that the dam could be regarded in plan as having been swung round on its left-hand abutment through an arc of a circle. The upstream alternative was found to be the more economical and was adopted.

Calculations for the dam structure were carried out by a cautious application of Tölke's method, which regards the dam as a solid of revolution with a constant external radius, and taking into account the elasticity of the rock. The conditions underlying the division of loads between horizontal arches and vertical cantilevers were determined, and a complete system of horizontal and vertical stresses was worked out for both the first-stage and the completed dam under three conditions: (a) hydrostatic pressure alone (with full reservoir), (b) hydrostatic pressure plus thermal stresses, and (c) hydrostatic pressure plus thermal stresses plus ice pressure. The maximum compressive stress was computed to be 56 kg/cm² and the maximum tensile stress 5 kg/cm².

These conclusions were supplemented by a site investigation to determine the modulus of elasticity of the rock and by model tests. The direction of dip of the strata is such that on the right bank the bedding planes follow roughly parallel to the line of the dam foundation whereas on the left bank they are more or less normal to it, and it was thought that the rock might behave differently on the two flanks. A heading was driven into each flank to reach a point about 40 m behind the abutment, and at this point a test chamber was built, about 5 m long, having a circular section 1·7 m in diameter lined with reinforced concrete in which were inserted four longitudinal joints. Reinforced-concrete plugs closed the ends of the chamber. Two remote-reading electrical extensometers sensitive to 0·002 mm were installed in each chamber on diameters at 45° to each other. Hydraulic pressure was then applied to the chamber and extensometer readings taken, the left-bank chamber being tested at pressures up to 29 kg/cm² and the right-bank chamber to 24 kg/cm². As was expected, the behaviour of the

right flank was much inferior to that of the left flank, and although grouting brought about a material improvement a significant difference remained. The results obtained are shown graphically in Fig. 6.

The anisotropic character of the foundation rocks was reproduced in the 1 : 50 scale models which were prepared at the ISMES laboratory in Bergamo. Tests on these models showed the effect of this characteristic of the foundations to be small. The model tests as a whole revealed that the calculations had been on the conservative side, the only point at which the stress was shown to be higher than calculated being at the arch of the crest. In fact, the maximum stress in the dam, which had been calculated at 54 kg/cm² (or 56 kg/cm² taking thermal and ice conditions into account) was found to be only 49 kg/cm², and in the light of this it was deemed safe to raise the crest of the first-phase dam by 4 m—from 1,898 m to 1.902 m—corresponding to an increase in loading of 10%.

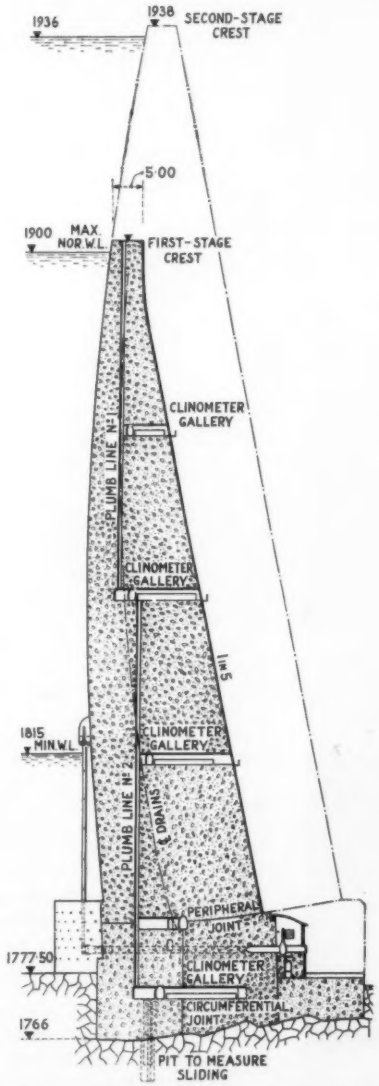

Fig. 9. Central section through Cancano dam, showing the second stage in outline

The type of dam contemplated obviously called for concrete of high quality, and a series of experiments was carried out to determine the most suitable type of cement. The cement chosen would need to have a high compressive strength, especially at 28 and at 90 days, and a reasonably low heat of hydration; furthermore it would require to be proof against attack by glacial or aggressive waters. These conditions were deemed to be fulfilled only by pozzolanic cements, and investigations were made into the characteristics of four such cements: a marine-type, a low-heat type, a normal pozzolanic cement, and a ferro-pozzolan cement. The individual cements were tested at Milan Polytechnic, the Istituto Sperimentale Stradale, and Padua University, and this work was succeeded by tests at the AEM laboratory on the resulting concretes.

Of the individual cements the marine pozzolan was found to have the highest average specific strength, at 175·1 kg/cm², followed by the normal pozzolan at 149·9 kg/cm², the ferro-pozzolan at 115·2 kg/cm², and the low-heat pozzolan at 110·9 kg/cm². The heat of hydration in relation to the specific strength was low for the marine and ferro-pozzolans and higher for the other two. Application of Bolomey's formula to determine the proportion of cement required to obtain a concrete having a resistance of 300 kg/cm² after 28 days, using 120 litres of water per m³ of concrete, gave the results shown on the left of Table III, indicating that the marine pozzolan was superior from this standpoint also, the proportion being only 198 kg/m³ and the heat liberated correspondingly low. It was considered, however, that for practical reasons the minimum permissible quantity of any cement should be 250 kg/m³ and by correspondingly adjusting the results as shown on the right of Table III the comparison becomes

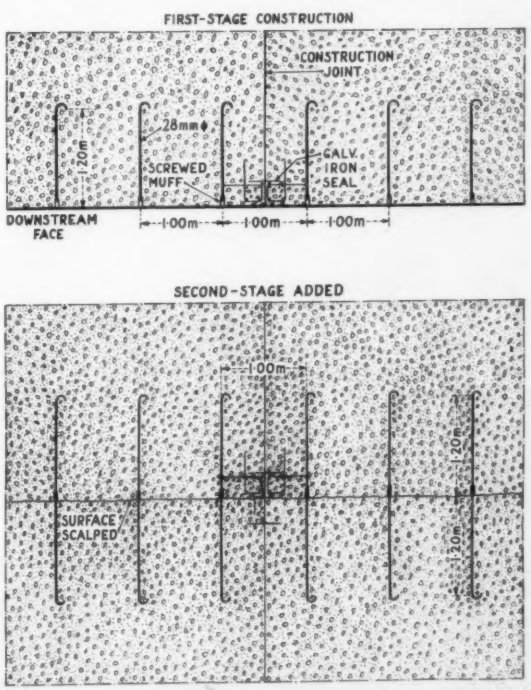

Fig. 10. Arrangement for grouting the second-stage concrete to the first

clear. The marine pozzolan needs the minimum quantity of cement—a position which it shares with the normal pozzolanic cement but its total heat of hydration is much less. In respect of heat of hydration its behaviour is roughly equivalent to that of the low-heat and the ferro-pozzolan cements but the resulting concrete, as already established, is much stronger. Thus from all points of view the marine-type pozzolan was obviously the correct choice.

Aggregate for the dam was obtained from a large deposit of detritus in the vicinity of Fraele, and was crushed, graded and washed to produce three sizes of material: coarse sand, 4–15 mm; gravel, 15–40 mm; shingle, 40–100 mm. The proportion of cement originally decided upon of 250 kg/m³ was reduced in the second year to 244 kg/m³ conjointly with the addition of an air-entraining agent, and later to 235 kg/m³. In the third year, when it had been decided to raise the crest by 4 m, the figure of 244 kg/m³ was reverted to. The 28-day strength of the concrete averaged 280 kg/cm², and test cores cut from the previous year's concrete in the actual dam possessed an average compressive strength of 465 kg/cm², a modulus of elasticity of 440,000 kg/cm², and a permeability under a pressure of 25 atm. for 10 days of nil.

Constructional Details of the Dam

A plan of the dam showing the first-stage construction, together with the associated discharge works, in full lines, and the final stage, chain-dotted, is given in Fig. 8; Fig. 9 shows the central section of the dam as now constructed, with the ultimate section chain-dotted. The dam proper rests on a concrete saddle, which is enlarged in the throat of the gorge to form a plug, contact between dam and saddle being through a peripheral joint. The leading dimensions of the dam in both first-phase and final stages are as follows:—

	First phase	Final phase
Crest level (m)	1,902	1,938
Maximum height above peripheral joint (m)	116	152
Maximum height above lowest foundation (m)	136	172
Crest external radius (m) ..	215·98	235·00
Crest chord (m)	343·4	380·6
Crest thickness at centre (m)	5·00	5·50
Base thickness at centre (m) ..	25·91	39·91
Volume of excavation (m³) ..	232,000	
Volume of concrete (m³) ..	525,500	

The dam is exactly symmetrical and is divided into 27 blocks each about 14 m wide. Between the blocks the joint surfaces were divided into panels by galvanised-iron sheet and grouted through a network of channels formed in the joint faces. Reinforcement is not used except in the vicinity of the inspection galleries and other internal openings. Drainage channels, 25 cm in diameter, are formed a few metres from the upstream face and descend to the inspection gallery which follows the circumferential joint.

To protect the upstream face from damage by ice it is faced with granite blocks. The downstream face is left exposed and will ultimately be scalped to make a good joint with the new concrete when the dam is raised to its final height. Further to ensure the soundness of this joint a series of hooked tiebars has been cast into the first-stage dam so that their ends are flush with the downstream face, these ends being screwed and protected by temporary caps. When the wall of the final stage is being brought up these caps will be replaced by screwed muffs into which will be inserted a mating series of tiebars which will be cast into the new wall. The arrangement is shown in Fig. 10. The purpose of these tiebars is not to afford design strength to the dam but to enable the joint between the old and the new concrete to be grouted without any risk of forcing the faces apart.

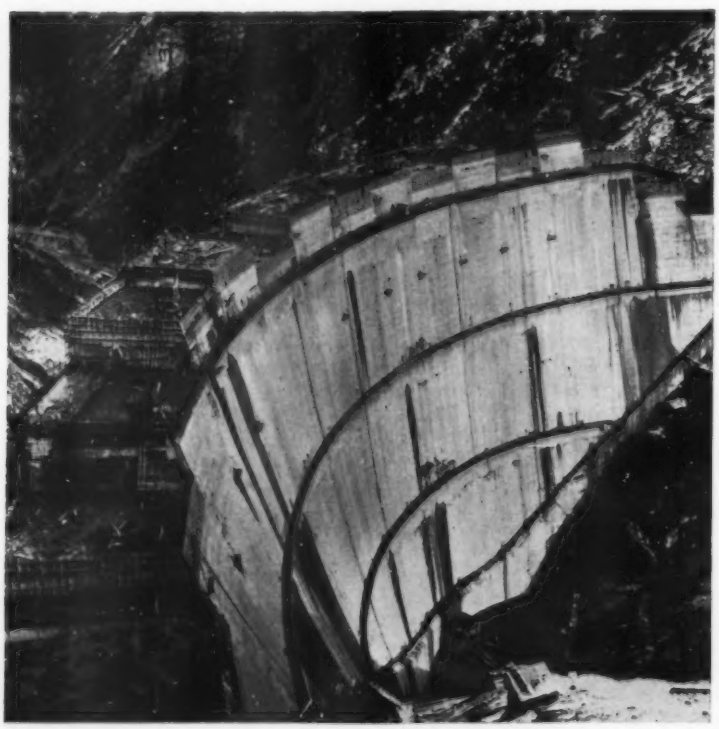

Fig. 11. Downstream view of the dam during construction

In the first-stage construction grouting was applied to the consolidation of the rocks at the abutments, the formation of an impervious curtain under the upstream face of the dam, the formation of a secondary curtain behind the main curtain, and the sealing of the joint between the concrete saddle and the rock. Altogether, 56,200 m of 35–45 mm holes were drilled, but the amount of cement absorbed was only 11·4 kg/m.

Discharge and Intake Works

The discharge facilities provided for the first-stage construction comprise a scour culvert, a bottom discharge, an intermediate discharge, and an uncontrolled spillway. The locations of these works can be seen in Fig. 8.

The scour culvert consists of a steel tube, 1·2 m in diameter, passing through the concrete plug in

Fig. 12. View of the three dams forming the Cancano and San Giacomo reservoirs. The new Cancano dam under construction is in the foreground, the old Cancano dam (now submerged) in the centre, and the San Giacomo dam in the distance

the throat of the gorge, and controlled by two valves in series.

As will be seen from Fig. 8 the bottom discharge is located in the right bank and consists of a branch taken off from the left-hand intake tunnel, which has been enlarged up to the point of bifurcation to carry the flow to the discharge (75 m^3/sec) as well as that to the turbines (40 m^3/sec). The intake tunnel has a diameter of 4·40 m up to the bifurcation, and the discharge branch is 3·25 m in diameter except for a venturi section, where it reduces to a rectangular opening of 1·80 by 1·10 m to accommodate the twin sliding gates.

The intermediate discharge is located in the left bank and consists of a tunnel 3·0 m in diameter, having a venturi section and a pair of sliding gates identical with those for the bottom discharge.

The surface spillway has been cut in the left bank, and comprises a sill 47 m long placed at an elevation of 1,900 m behind which a trough-shaped channel directs the water to an inclined shaft leading to a tunnel 3 m in diameter. When the dam is carried to its final height this spillway will naturally cease to be usable as such, and it will therefore be converted into a high-level relief discharge. During the construction of the spillway an adit was cut, as seen in Fig. 8, to enable the tunnel to be driven, and this adit will be reopened and fitted with a pair of sluice gates, the present spillway being blocked off. A new spillway will then be cut higher up the side of the bank, as indicated in Fig. 8.

At the present stage the discharge works have the following capacities: bottom discharge 63 m^3/sec, intermediate discharge 55 m^3/sec, surface spillway

100 m³/sec. This makes possible an emergency discharge of 218 m³/sec, but normally the discharge is kept within 100 m³/sec to avoid overflooding of the river downstream of the dam.

Earlier in this article it was explained that Premadio station has been built in the first instance for approximately half its ultimate capacity. To afford the maximum security and continuity of operation the hydraulic works of the two halves of the station have been kept entirely separate, so that each half has its own intake, supply tunnel and surge chamber. These works, of course, have been completed for the first stage; for the second stage the intake and supply tunnel have been completed as far as the valve chamber, and a pilot heading for the remainder of the tunnel carried forward a distance of 1,250 m.

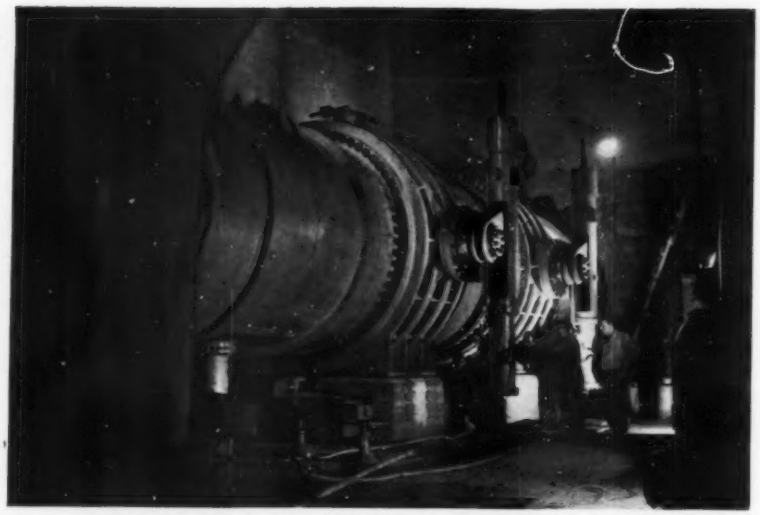

Fig. 14. Head-valve chamber for Premadio station

The supply works for the first stage consist of a horizontal pressure tunnel, 3,467 m long from intake to surge chamber, succeded by a valve chamber controlling the flow to a steel penstock laid freely in an inclined shaft. The pressure tunnel is circular and has a finished diameter of 3·42 m. It has been concrete lined to a thickness of 30 cm, followed by high-pressure grouting behind the lining, and has been designed to withstand an ultimate head of 170 m.

The surge chamber comprises a lower compensating chamber and an upper expansion chamber connected by a shaft 4 m in diameter and 138 m high. The compensating chamber is in the form of a trident and has a capacity of 1,900 m³. From the cross of the trident an air shaft 1·5 m in diameter rises at 45° to join the main vertical shaft. This vertical shaft terminates in a differential-type overflow within the upper expansion chamber, which at present has a capacity of 1,600 m³ but will be enlarged to 2,600 m³ when the dam is raised to its full height.

Fig. 13. Premadio penstock tunnel and penstock

Two butterfly valves, 2·8 m in diameter, provided with air relief valves and arranged for automatic closing in the event of excess velocity of flow, protect the penstock. The penstock shaft is 4·5 m wide and 4·1 m high and is inclined at 45°. Concrete saddles support the penstock, which consists of a straight length of 853 m connecting upper and lower horizontal sections, respectively 26 m and 9 m long. The upper 160 m of the penstock consists of welded steel plate ranging in thickness from 20 to 30 mm and having a diameter of 2·6 m. The lower portion is of hooped construction, the diameter decreasing from 2·6 to 2·3 m and the metal thickness ranging up to 38 mm for the plate and 56 mm for the hoops. Finally the penstock divides into four sections to supply the two turbines forming the first half of the installation. This distributor is also of hooped construction and is concreted solid in the rock. Fig. 13 is a view in the penstock tunnel, and the head valves can be seen in Fig. 14.

Contractors

The contractors engaged to build the dam, the side-intake tunnels and the diversion tunnel, were Impresa Lodigiani, Milan, and the construction of the supply tunnel and surge chamber was the work of Impresa Filli Tettamanti, Milan. The penstock tunnel was driven by Sta. Italiana per Condotte d'Aqua, Rome. All the sluice gates with their operating mechanisms were made by Acciaieria e Tubificio di Brescia except for the scour sluice and its associated valves, which were supplied by Officine Calzoni di Bologna.

In our next article we shall describe Premadio station, and our third and final article will be devoted to Grosio station.

(To be continued)

Specific-Energy and Force Equations in Open-Channel Flow

The equations in this article can be written in similar forms, for which a tabular solution is available. The analysis is extended from that for the rectangular channel to that for trapezoidal and parabolic sections

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GLOSSARY OF TERMS

These are in the order of their appearance. The subscript c refers to conditions at the critical depth.

E_s=specific energy of stream referred to the channel bed (ft)
y, d=water depth (ft)
v=mean velocity of flow (ft/sec)
g=acceleration due to gravity (ft/sec^2)
Q=total discharge (cusecs)
A=area of water cross section (ft^2)
b_w=width of water surface (ft)
b'_w=equivalent width of rectangular channel, i.e. same depth and area as non-rectangular shape (ft)
b=bottom width of trapezoidal channel (ft)
n=side slopes of channel are given as 1 on n
a=constant in the parabolic formula $b_w=ay^{\frac{1}{2}}$
F=total force in a stream (lb)
w=unit weight of water (i.e. 62·4 lb/ft^3)
h=depth of centre of gravity of water cross section (ft)
k=fraction dependent on shape of water cross section
dx=portion of channel length in backwater calculations (ft)
s_o=bed slope of channel
s_f=slope of water surface over section dx of channel
y_o=normal depth of channel.

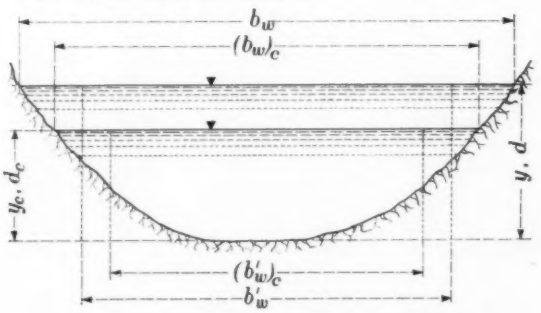

Fig. 1. Generalised section of channel

Specific Energy

The specific energy of a stream is given by

$$E_s=y+\frac{v^2}{2g}=y+\frac{Q^2}{2gA^2}$$

dividing through by y_c, we get

$$\frac{E_s}{y_c}=\frac{y}{y_c}+\frac{Q^2}{2gA^2y_c}$$

but also $\frac{Q^2}{g}=\frac{A^3{}_c}{(b_w)_c}=\frac{y_c{}^3(b'_w)_c{}^3}{(b_w)_c}$ (see Fig. 1)

thus $$\frac{E_s}{y_c}=\frac{y}{y_c}+\frac{1}{2}\left[\frac{y_c{}^3(b'_w)_c{}^3}{(b_w)_cA^2y_c}\right]$$

or $$\frac{E_s}{y_c}=\frac{y}{y_c}+\frac{1}{2}\left(\frac{y_c}{y}\right)^2\left[\frac{(b'_w)_c{}^3}{(b_w)_c(b'_w)^2}\right] \quad \ldots (1)$$

For the rectangular channel this reduces to[1]

$$\frac{E_s}{y_c}=\frac{y}{y_c}+\frac{1}{2}\left(\frac{y_c}{y}\right)^2 \quad \ldots (2)$$

a graph of which appears in Fig. 2.

Since it is not always convenient to read small variations in parameters from a graph a table of values for $\frac{E_s}{y_c}$ for small increments in $\frac{y}{y_c}$ has recently been published[2].

But if shapes of channel other than rectangular are

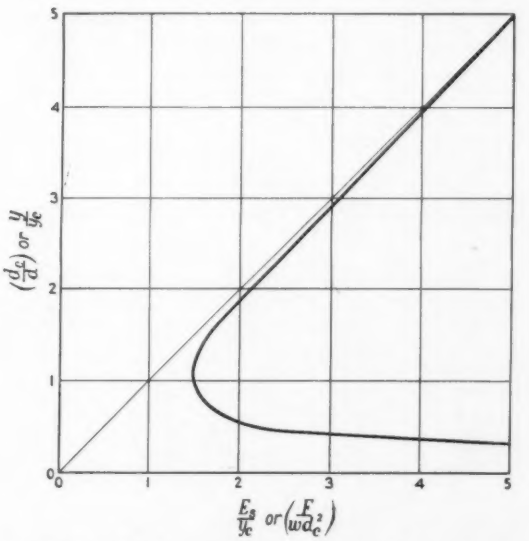

Fig. 2. Graph of generalised energy and force equations for rectangular channels

to be treated, allowance must be made to apply the bracketed term or "shape factor" in equation (1). As this only affects one term of $\frac{E_s}{y_c}$ it seems more convenient to tabulate $\frac{1}{2}\left(\frac{y_c}{y}\right)^2$ for various values of $\left(\frac{y}{y_c}\right)$ (see Table I). In this way either term (see section on the force equation) can be multiplied by a factor and $\frac{E_s}{y_c}$ is obtained by a simple addition.

TABLE I

$\frac{1}{2}\left(\frac{y_c}{y}\right)^2$ as a function of y/y_c

or $\frac{1}{2}\left(\frac{d}{d_c}\right)^2$ as a function of d_c/d

or $\frac{y/y_c}{d_c/d}$	0·00	0·01	0·02	0·03	0·04	0·05	0·06	0·07	0·08	0·09
0·0	—	5000	1250	555·6	312·5	200·0	138·8	102·0	78·13	61·73
0·1	50·0	41·32	34·73	29·58	25·51	22·22	19·53	17·30	15·43	13·85
0·2	12·50	11·34	10·33	9·452	8·680	8·000	7·396	6·859	6·378	5·945
0·3	5·556	5·203	4·883	4·591	4·325	4·082	3·858	3·652	3·463	3·287
0·4	3·125	2·974	2·834	2·704	2·583	2·469	2·363	2·264	2·170	2·082
0·5	2·000	1·922	1·849	1·780	1·715	1·653	1·594	1·539	1·486	1·436
0·6	1·389	1·344	1·301	1·260	1·221	1·183	1·148	1·114	1·081	1·050
0·7	1·020	0·992	0·964	0·938	0·913	0·889	0·866	0·843	0·822	0·801
0·8	0·781	0·762	0·744	0·726	0·709	0·692	0·676	0·661	0·646	0·631
0·9	0·617	0·604	0·591	0·578	0·566	0·554	0·542	0·531	0·521	0·510
1·0	0·500	0·490	0·481	0·471	0·462	0·454	0·445	0·437	0·429	0·421
1·1	0·413	0·406	0·399	0·392	0·385	0·378	0·371	0·365	0·359	0·353
1·2	0·347	0·341	0·335	0·330	0·325	0·320	0·315	0·310	0·305	0·300
1·3	0·296	0·291	0·287	0·283	0·279	0·274	0·270	0·266	0·263	0·259
1·4	0·255	0·251	0·247	0·244	0·241	0·238	0·235	0·231	0·228	0·225
1·5	0·222	0·219	0·216	0·213	0·210	0·207	0·204	0·201	0·198	0·195
1·6	0·196	0·193	0·191	0·188	0·186	0·184	0·182	0·179	0·177	0·175
1·7	0·173	0·171	0·169	0·167	0·165	0·163	0·161	0·160	0·158	0·156
1·8	0·154	0·152	0·151	0·149	0·148	0·146	0·144	0·143	0·142	0·140
1·9	0·138	0·137	0·136	0·134	0·133	0·132	0·130	0·129	0·128	0·126
2·0	0·125	0·124	0·123	0·122	0·120	0·119	0·118	0·117	0·116	0·115
2·1	0·114	0·113	0·112	0·111	0·110	0·109	0·108	0·107	0·106	0·105
2·2	0·104	0·103	0·102	0·101	0·100	0·099	0·098	0·097	0·096	0·095
2·3	0·095	0·094	0·093	0·092	0·091	0·090	0·090	0·089	0·088	0·088
2·4	0·087	0·086	0·086	0·085	0·084	0·083	0·083	0·082	0·081	0·081
2·5	0·080	0·079	0·079	0·078	0·077	0·076	0·076	0·075	0·075	0·074
2·6	0·074	0·073	0·073	0·072	0·072	0·071	0·071	0·070	0·070	0·069
2·7	0·069	0·068	0·068	0·067	0·067	0·066	0·066	0·065	0·065	0·064
2·8	0·064	0·063	0·063	0·062	0·062	0·061	0·061	0·060	0·060	0·059
2·9	0·059	0·059	0·058	0·058	0·058	0·057	0·057	0·057	0·056	0·056
3·0	0·056	0·055	0·055	0·055	0·054	0·054	0·054	0·053	0·053	0·053
3·1	0·052	0·052	0·052	0·051	0·051	0·051	0·050	0·050	0·050	0·049
3·2	0·049	0·049	0·048	0·048	0·048	0·047	0·047	0·047	0·046	0·046
3·3	0·046	0·046	0·045	0·045	0·045	0·045	0·044	0·044	0·044	0·044
3·4	0·043	0·043	0·043	0·043	0·042	0·042	0·042	0·042	0·041	0·041
3·5	0·041	0·041	0·040	0·040	0·040	0·040	0·039	0·039	0·039	0·039
3·6	0·039	0·038	0·038	0·038	0·038	0·038	0·037	0·037	0·037	0·037
3·7	0·037	0·036	0·036	0·036	0·036	0·036	0·035	0·035	0·035	0·035
3·8	0·035	0·034	0·034	0·034	0·034	0·034	0·033	0·033	0·033	0·033
3·9	0·033	0·033	0·032	0·032	0·032	0·032	0·032	0·032	0·031	0·031
4·0	0·031	0·031	0·031	0·031	0·030	0·030	0·030	0·030	0·030	0·030
4·1	0·030	0·029	0·029	0·029	0·029	0·029	0·029	0·029	0·028	0·028
4·2	0·028	0·028	0·028	0·028	0·028	0·028	0·027	0·027	0·027	0·027
4·3	0·027	0·027	0·027	0·027	0·026	0·026	0·026	0·026	0·026	0·026
4·4	0·026	0·026	0·026	0·025	0·025	0·025	0·025	0·025	0·025	0·025
4·5	0·025	0·025	0·024	0·024	0·024	0·024	0·024	0·024	0·024	0·024
4·6	0·024	0·023	0·023	0·023	0·023	0·023	0·023	0·023	0·023	0·023
4·7	0·023	0·022	0·022	0·022	0·022	0·022	0·022	0·022	0·022	0·022
4·8	0·022	0·022	0·021	0·021	0·021	0·021	0·021	0·021	0·021	0·021
4·9	0·021	0·021	0·021	0·021	0·020	0·020	0·020	0·020	0·020	0·020

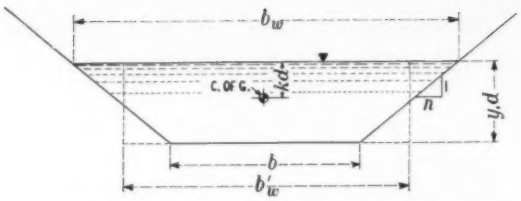

Fig. 3. A typical trapezoidal channel

The shape factor $\left[\frac{(b'_w)_c^3}{(b_w)_c(b'_w)^2}\right]$ can easily be evaluated for the trapezoidal and triangular channels from the geometry of the section. From Fig. 3 it is readily seen that $b'_w = b + ny$ and that $b_w = b + 2ny$, the critical values being determined for $y = y_c$. Fig. 4 is a graph to assist such calculations.

It can be shown, for a parabolic channel, where $b_w = ay^{\frac{1}{2}}$, that $b'_w = \frac{2}{3} b_w = \frac{2}{3} ay^{\frac{1}{2}}$ so that the shape factor becomes $\left[\frac{2}{3}\frac{y_c}{y}\right]$. This has been derived in a slightly different manner elsewhere[3].

For any other shape of channel a scaled drawing would provide values of b_w and b'_w near enough to compute the shape factor.

EXAMPLE 1

A trapezoidal channel of bottom width 20 ft and side slopes 1 on 2 has water flowing in it at a depth of 12 ft when the discharge is 4,000 cusecs. A bridge with two piers 3 ft wide runs across the channel; determine the depth of the water under the bridge.

The details are given in Fig. 5.

Now $\frac{Q}{g^{\frac{1}{2}}} = \frac{4{,}000}{(32{\cdot}2)^{\frac{1}{2}}} = 706$ and hence $y_c = 8{\cdot}1$ (from graphs published elsewhere[4]).

From Fig. 5 or Fig. 4 we have

$$(b'_w)_c = 20 + 16{\cdot}2 = 36{\cdot}2$$
$$(b_w)_c = 20 + 32{\cdot}4 = 52{\cdot}4$$
$$b_w = 20 + 24 = 44$$

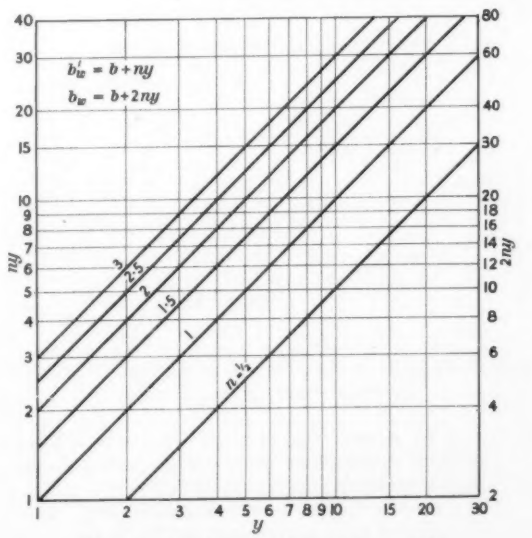

Fig. 4. Graph to evaluate b'_w *and* b_w

from which we get a shape factor of 0·47 (say 0·5).

Upstream of the bridge we have

$$\frac{E_s}{y_c} = \frac{y}{y_c} + \frac{1}{2}\left(\frac{y_c}{y}\right)^2 \quad [0{\cdot}5]$$
$$= \frac{12}{8{\cdot}1} \quad (= 1{\cdot}48) + 0{\cdot}228 \quad [0{\cdot}5] \text{ from Table I}$$
$$= 1{\cdot}594.$$

Under the bridge the cross section is equivalent to a trapezoidal channel with a bottom width of 14 ft for which[4] $y_c = 9{\cdot}0$ ft.

We now have $(b'_w)_c = 14 + 18 = 32$
$(b_w)_c = 14 + 36 = 50$
$b'_w = 14 + 24 = 38$ (near enough)
giving a shape factor of 0·45, say 0·5.

Assuming no loss of specific energy at the bridge we have $1{\cdot}594 \left(\frac{8{\cdot}1}{9{\cdot}0}\right) = 1{\cdot}435 = 1{\cdot}28 + 0{\cdot}305$ [0·5] from inspection of Table I, and hence $\frac{y}{9{\cdot}0} = 1{\cdot}28$, i.e., $y = 11{\cdot}5$ ft.

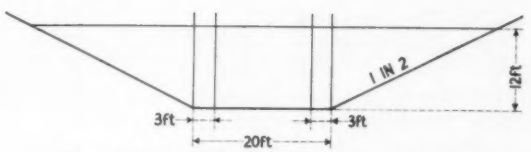

Fig. 5. Detail of channel for example 1

Force Equation

The total force on a stream is the sum of the dynamic and hydrostatic forces, namely

$$F = \frac{wQv}{g} + whA$$
$$\text{or } \frac{F}{w} = \frac{Q^2}{gA} + hA$$

To assist in the use of Table I it is convenient to symbolise depths by d rather than y, hence, dividing through by d^2_c we get

$$\frac{F}{wd^2_c} = \frac{Q^2}{gAd^2_c} + \frac{hA}{d^2_c}$$

but also $\frac{Q^2}{g} = \frac{A^3_c}{(b_w)_c}$ and $A = b'_w d$

hence, if $h = kd$ where k is a fraction dependent on the cross section, we have

$$\frac{F}{wd^2_c} = \frac{A^3_c}{(b_w)_c A d^2_c} + \frac{(kd)b'_w d}{d^2_c}$$
$$= \frac{(b'_w)^3_c d^3_c}{(b_w)_c d^2_c (b'_w) d} + \frac{2kb'_w}{2}\left(\frac{d}{d_c}\right)^2$$

$$\text{or } \frac{F}{wd^2_c} = \left(\frac{d_c}{d}\right)\left[\frac{(b'_w)^3_c}{(b_w)_c(b'_w)}\right] + \frac{1}{2}\left(\frac{d}{d_c}\right)^2 \quad [2k(b'_w)] \quad \ldots (3)$$

which for the rectangular channel reduces to[1]

$$\frac{F}{wd^2_c} = \left(\frac{d_c}{d}\right) b_w + \frac{1}{2}\left(\frac{d}{d_c}\right)^2 b_w \quad \ldots (4)$$

This force F is the total force over the cross section, and per unit width appears the same as equation (2) in Fig. 2.

Francis[1] first recognised the similarity of this equation to the dimensionless specific-energy equation discussed above. More recently it was derived again[3]. Table I serves to determine $\frac{1}{2}\left(\frac{d}{d_c}\right)^2$ for various values of $\left(\frac{d_c}{d}\right)$.

The allowance for non-rectangular shapes consists of two terms, one similar but not identical to that used in equation (1), and the other containing a proportion of depth giving the distance from the water surface down to the centre of gravity of the water cross section.

For the trapezoidal channel (see Fig. 3) we have

$$k=\frac{(b_w+2b)}{3(b_w+b)}=\frac{(2b'_w+b)}{3(2b'_w)}$$

$$\therefore\ [2k(b'_w)]=\frac{2b'_w+b}{3}=\frac{2b+2ny+b}{3}=b+\frac{2}{3}\ ny.$$

The value of ny can be obtained from Fig. 4 or the inspection of the cross section.

For the parabolic channel, knowing $b_w=ad^{\frac{1}{2}}$ and $b'_w=\frac{2}{3}\ b_w$, the first bracketed term becomes $\left[\frac{4}{9}\ \frac{ad_c}{d^{\frac{1}{2}}}\right]$. Also $k=\frac{2}{5}$ and hence the second term becomes $\left[\frac{8}{15}\ ad^{\frac{1}{2}}\right]$. The equation for the parabolic channel thus becomes

$$\frac{F}{wd^2{}_c}=\left(\frac{d_c}{d}\right)\left[\frac{4}{9}\ \frac{ad_c}{d^{\frac{1}{2}}}\right]+\frac{1}{2}\left(\frac{d}{d_c}\right)^2\left[\frac{8}{15}\ ad^{\frac{1}{2}}\right]\quad\ldots\ (5)$$

which has been derived elsewhere in a slightly different form[3].

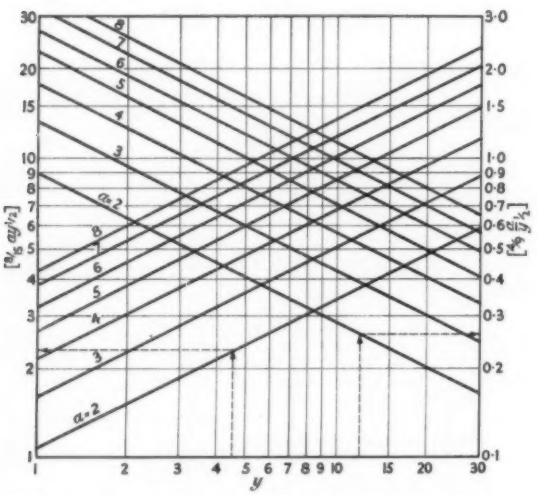

Fig. 6. Graph to evaluate shape factors in equation 5

Fig. 6 should assist in computing the square-bracketed terms in equation (5). Note that only the value of $\frac{4}{9}\ \frac{a}{d^{\frac{1}{2}}}$ is given in the figure.

EXAMPLE 2

In a parabolic channel ($a=4$) a discharge of 1,000 cusecs is flowing at a depth of 6 ft, which is below the critical depth. To what height must the downstream level rise before a hydraulic jump forms at this point?

Now $\frac{Q}{g^{\frac{1}{2}}}=\frac{1{,}000}{(32{\cdot}2)^{\frac{1}{2}}}=176$ from which[4] $d_c=9{\cdot}1$.

From Fig. 6 for $y=6$ we have

$\left[\frac{4}{9}\ \frac{ad_c}{d^{\frac{1}{2}}}\right]=0{\cdot}72\times9{\cdot}1=6{\cdot}55$ and $\left[\frac{8}{15}\ ad^{\frac{1}{2}}\right]=5{\cdot}1$.

Hence upstream of the jump the force in the stream is given by $\frac{F}{wd^2{}_c}=\left(\frac{d_c}{d}\right)[6{\cdot}55]+\frac{1}{2}\left(\frac{d}{d_c}\right)^2[5{\cdot}1]$

$=\left(\frac{9{\cdot}1}{6}\right)[6{\cdot}55]+(0{\cdot}216)\ [5{\cdot}1]$ from Table I

$=9{\cdot}96+1{\cdot}11=11{\cdot}07.$

This force term must be the same downstream of the jump. Solving by trial and error we know that the depth must be greater than critical.

Try $d=10$ ft, then from Fig. 6 and Table I

$$11{\cdot}07=\left(\frac{9{\cdot}1}{10}\right)[0{\cdot}565\times9{\cdot}1]+(0{\cdot}604)\ [6{\cdot}7]$$

$$=4{\cdot}68+4{\cdot}04=8{\cdot}72\neq11{\cdot}07.$$

Try $d=12$ ft, then

$$11{\cdot}07=\left(\frac{9{\cdot}1}{12}\right)[0{\cdot}515\times9{\cdot}1]+(0{\cdot}866)\ [7{\cdot}4]$$

$$=9{\cdot}96\neq11{\cdot}07.$$

Try $d=13$ ft, then

$$11{\cdot}07=\left(\frac{9{\cdot}1}{13}\right)[0{\cdot}5\times9{\cdot}1]+(1{\cdot}02)\ [7{\cdot}7]$$

$$=11{\cdot}04\doteq11{\cdot}07.$$

Thus the required depth is 13 ft.

Backwater Curves

It can easily be shown that $dx=\frac{E_2-E_1}{S_o-S_f}$ with the notation as in the glossary of terms. The friction factor S_f can be obtained from graphs available[4] and the specific energies from data provided herein or in reference 2. By computing the specific energies at two points and the friction factor S_f, the length dx of the channel necessary to effect such changes can be determined. The addition of several lengths provides a backwater profile of the water surface[4, 5].

EXAMPLE 3

In a long trapezoidal channel, with bottom width 25 ft and side slopes 1 on 1·5, a discharge of 800 cusecs occurs. If the bed slope is 0·0004 and the downstream stage is 12 ft, what is the distance upstream to where the depth is within 5% of normal? (Manning's $n=0{\cdot}025$.)

Fig. 7 shows the detail of the problem.

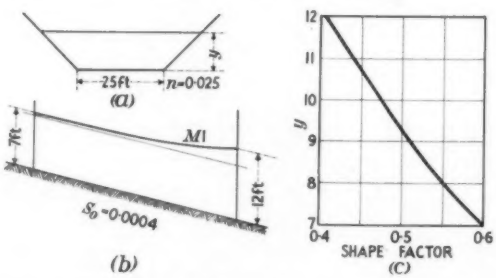

Fig. 7. Details of the channel in example 3, and its derived shape-factor diagram

It can be shown[4] that $y_o=6{\cdot}75$ ft and $y_c=3{\cdot}0$ ft.

The slope is therefore mild and the water surface is concave upwards (i.e., curve $M1$).

The shape factor can be graphed as in Fig. 7c and values inserted that correspond to the stage. Since computations to within 5% of y_o are required, com-

1	2	3(a)	4(b)	5(c)	6(d)	7(e)	8(f)	9	10(g)	11(h)
y	$\left(\frac{y}{y_c}\right)$	$\frac{1}{2}\left(\frac{y_c}{y}\right)^3$	S.F.	S.F. × Col. 3	$\frac{E_s}{y_c}$	$\frac{E_2-E_1}{y_c}$	k	$S_f \times 10^3$	$(S_o-S_f)\times 10^3$	$\frac{dx}{y_c}$
7·0	2·33	0·091	0·60	0·055	2·385		1060	0·35	0·05	
						0·066				1016
7·2	2·40	0·087	0·59	0·051	2·451		1110	0·32	0·08	
						0·095				905
7·5	2·50	0·080	0·57	0·046	2·546		1200	0·27	0·13	
						0·163				1050
8·0	2·67	0·070	0·55	0·039	2·709		1340	0·22	0·18	
						0·155				755
8·5	2·83	0·062	0·53	0·033	2·863		1500	0·17	0·23	
						0·166				664
9·0	3·0	0·056	0·51	0·029	3·029		1700	0·13	0·27	
						0·322				1140
10·0	3·33	0·045	0·47	0·021	3·351		2050	0·093	0·307	
						0·335				1040
11·0	3·67	0·037	0·44	0·016	3·686		2450	0·064	0·336	
						0·327				947
12·0	4·0	0·031	0·41	0·013	4·013		2900	0·045	0·355	
										Total 7517

(a) From Table I.
(b) Shape factor from Fig. 7c.
(c) Correction made for shape.
(d) Col. 2 plus col. 5.
(e) Downstream section minus upstream and therefore positive.
(f) Values of k in $S_f=\frac{Q^2n^2}{k^2}$ obtainable in Reference 4.
(g) Positive (it is possible to have both col. 7 and col. 10 negative).
(h) Col. 7 ÷ average of two consecutive values in col. 10. dx should always be positive.

mence at $y=7{\cdot}0$, as shown in the adjoining table.

Thus the length of the backwater curve is $d_c \times 7{,}517 = 22{,}600$.

It is evident in example 3 that the term $\frac{1}{2}\left(\frac{y_c}{y}\right)^2$ is small and that the influence of the shape factor on $\frac{E_s}{y_c}$ is negligible. For a rough calculation of dx, therefore, it could be omitted. This would not be the case where the depth is approaching or is below the critical depth.

It may be noted that in Table I critical conditions exist for $\frac{E_s}{y_c}$ when $\frac{y}{y_c} \leqq 1$ and for $\frac{F}{wd^2_c}$ when $\frac{d_c}{d} \geqq 1$.

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4. R. SILVESTER, "New Graphs Aid Open-Channel Calculations," WATER POWER, 1954, p. 339.
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Film on Hydraulics Research

An enjoyable 16 mm film, entitled "The Ways of Water," was recently exhibited at the Central Office of Information in London. This is a colour film depicting the facilities and work of the Hydraulics Research Station, which is a division of the Departmen of Scientific and Industrial Research, and is situated at Wallingford, Berkshire, England.

The film, which has an explanatory soundtrack and runs for about 28 minutes, is purposely pitched at a reasonably high technical, rather than at a popular level, and is intended for engineers and the technical and administrative staffs of organisations throughout the world that might be interested in the work of the Hydraulics Research Station. It is also directed towards university and technical-college students who might be contemplating joining the staff of the station. The film covers a wide variety of the station's activities, both in pure and applied hydraulics research, and includes scenes of such model studies as those of Hong Kong harbour and the wave-protection devices on the Mangla dam, besides views of investigations on site. The large proportion of work in hand for overseas clients, either directly or through consulting engineers, is a point that is effectively brought out by the film. Our own interest in the film proved to be such that we could have wished it to be longer.

"The Ways of Water" is to be produced with soundtracks in a variety of languages, and enquiries about the loan of the film should be made to the Central Film Library, Government Building, Bromyard Avenue, Acton, London, W.3.

Truck Mixers. Ransomes & Rapier Limited, Waterside Works, Ipswich, England, have produced a brochure that describes their hydraulic-drive truck mixers in the 4–9 cu yard range.

Flygt Pump Manufacturers. Stenberg Corporation AB, of Stockholm, manufacturers of the Flygt submersible electric pumps, have announced a change in the name of their company. The company will now be known as Flygt International AB.

Advances in Low-Head Machines

New possibilities in the development of low heads, whether for run-of-river or for tidal plants, are opened up by the advent of the large axial-flow bulb-type machine. Some typical large-capacity installations are described in this second article

By S. CASACCI

PART TWO

Upstream-Bulb Set at Beaumont-Monteux

THIS set, of 8,000 kW capacity, operates under a head ranging from 9·85 to 12·48 m. It has movable runner blades and guide vanes, and comprises the hydraulic circuit, the upstream spider, and the dismountable monobloc section (Fig. 8).

The hydraulic circuit includes two concrete sections (inlet and draft tube), fixed elements sealed into the concrete, and removable elements (alternator casing, guide-vane ring, bearing shrouding). To mount the set rigidly all parts except those which must be made removable for maintenance or replacement are embedded in concrete.

The upstream spider comprises a central ogive and three arms with a carefully designed profile. The uppermost arm affords access to the interior of the bulb and carries the service pipes and runner-blade control gear. Access to the bearing adjacent to the turbine runner is effected through the alternator rotor. The two lower arms are used for ventilation, the air being circulated by an independent fan through an external cooler. The upstream section of the central ogive is partitioned off to form a lubricating-oil tank, which is naturally cooled.

The monobloc section (Fig. 9) comprises the fixed guide ring, which forms the second support for the section, the Alsthom alternator, the shaft and its bearings, the thrust bearing, the shaft coupling, the runner, and the movable guide vanes. The alternator rotor is welded directly to a one-piece shaft which is carried on two spherically mounted bearings. The thrust bearing, which is of the tilting-pad type, is

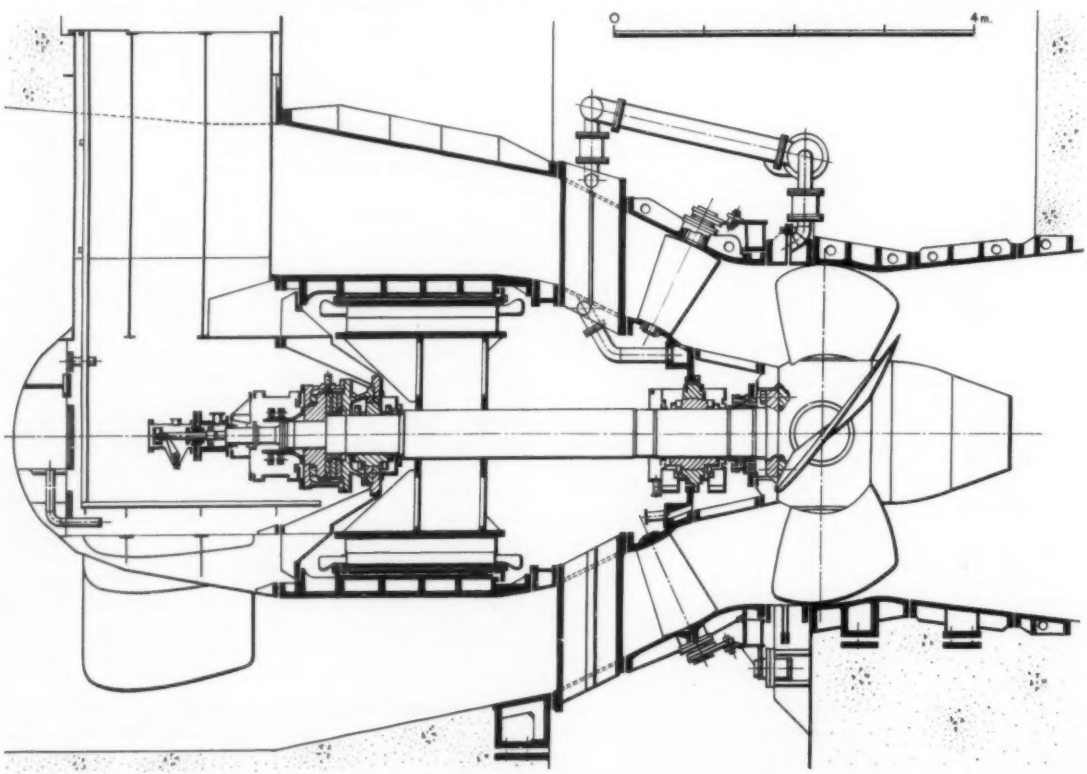

Fig. 8. The 8,000-kW upstream-bulb installation at Beaumont-Monteux

located at the upstream end of the shaft and incorporates a counterthrust element. A carbon sealing gland is provided at the point where the shaft leaves the bulb.

The runner has four movable but non-reversible blades and is overhung. Its hub contains the blade servomotor, and the oil pipes and the hollow control rod pass through the centre bore of the shaft to distribution channels in the hub of the upstream spider.

The distributor, which is conical, comprises a set of movable vanes mounted between two concentric spherical rings carrying the bushes and seals. The vanes are connected through a system of bars and ball-and-socket mounted links to a regulating ring moved by two vertical servomotors under the control of the governing system.

The monobloc section just described is assembled vertically on the floor, turned over and lowered into place.

The set is provided with all the usual accessories and safety devices.

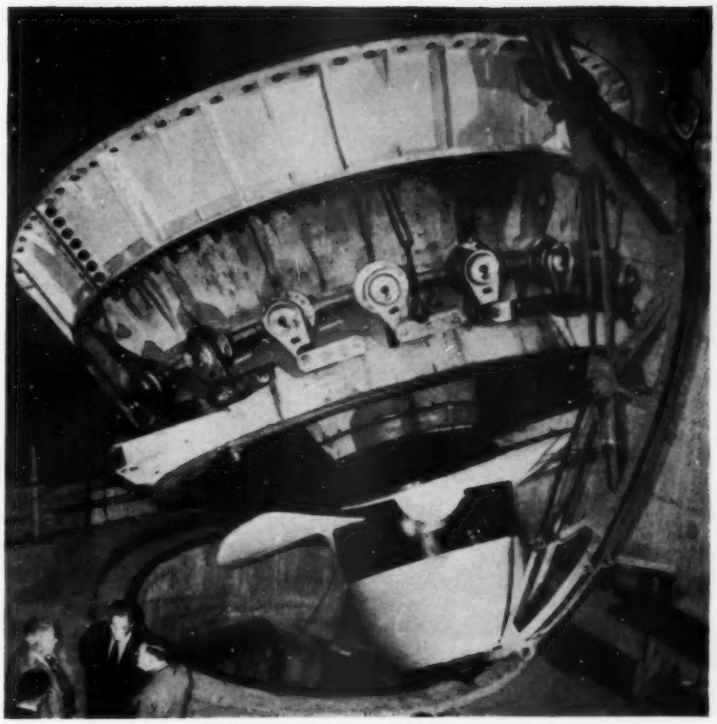

Fig. 9. Monobloc section of the Beaumont-Monteux set

Downstream-Bulb Set at Argentat*

This set, of 14,400 kW capacity, operates as a turbine under heads ranging from 8 to 16·5 m, but for experimental purposes only it can be arranged to operate as a pump. It has movable blades.

As will be seen from Fig. 10, apart from the conduit itself, the assembly, reading from upstream to downstream, comprises the access shaft and tunnel, the fixed guide vanes, the movable guide vanes, a conical shrouding housing the thrust bearing, the runner, the upstream fairing of the alternator casing, the alternator itself, and the downstream fairing.

The hydraulic conduit, which needs little description, comprises sections in concrete, embedded parts, and sections that are removable to permit dismantling the turbine or the alternator. To simplify the civil engineering the whole of the conduit between the runner throat ring and the downstream fairing of the alternator casing is steel lined.

The access shaft and tunnel are entirely of steel plate; they are embedded in the central pillar of the intake and accommodate various cables and pipes leading to the turbine. A junction ring connects the tunnel to the inner flange of the distributor, and transmits to the central pillar the hydraulic thrusts exerted on the runner and on the inner pins of the guide vanes. Within this junction ring are the pumps and cooling coils for the thrust-bearing lubrication, leakage pumps, oil-distribution channels and the pilot alternator.

The fixed guide wheel, which is concreted in place, carries the thrust bearing through the intermediary

* See WATER POWER, April 1958, p. 156.

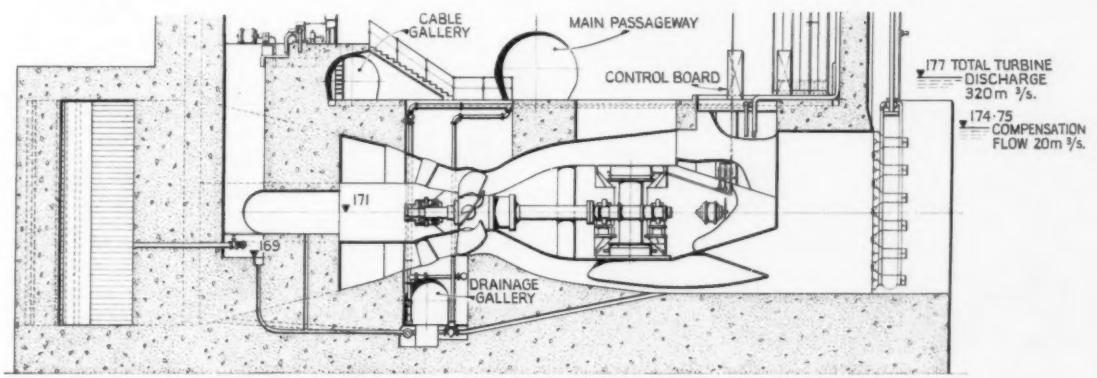

Fig. 10. The 14,400-kW downstream-bulb installation at Argentat

Fig. 11. Distributor for the Argentat set

of the junction ring and thus forms the upstream support for the shaft. The junction ring houses the upstream carbon gland as well as the bearing.

The distributor (Fig. 11) is similar to a normal conical distributor and is operated by an external regulating ring actuated by two vertical oil servomotors controlled by the governor. Each guide vane is locked hydraulically to avoid risk of vibration when the machine is functioning as a pump.

The thrust bearing consists of tilting pads carried on a spherically mounted thrust ring and includes provision for back-thrust. It is designed to enable the machine to function either as a turbine or as a pump, i.e. to rotate in either direction.

The runner is equipped with non-reversible movable blades and forms an integral part of the shaft assembly. Its hub, coupled to the upstream shaft, is followed by the servomotor body, which in turn is coupled to the intermediate shaft connecting the turbine to the alternator.

The upstream fairing of the alternator carries the downstream carbon gland and the brake. Access to the inside of this fairing can be gained either by passing through the alternator rotor if the set is under water, or through a manhole if the conduit is blanked off and dewatered. The alternator, supplied by Alsthom, is complete with its shaft and two bearings, and is carried on two steel supports placed across the part of the conduit forming the turbine draft tube. The profiles of these supports were carefully determined by model tests. The downstream fairing, the outer surface of which is used for alternator cooling, encloses the ventilating fan. From this fairing rises a vertical streamlined shaft which helps to locate the set, and the various compartments of which are used for access to the alternator and for cables and pipes.

The foregoing brief description shows that the Argentat machine is radically different from that at Beaumont-Monteux. On the other hand, the Cambeyrac machine, now to be described, is very similar to that at Argentat.

Cambeyrac Downstream-Bulb Unit

The design of this set, which develops 5,000 kW under a head of 10·75 m and is capable of functioning either as a turbine or as a pump in either direction of flow, raised problems similar to those associated with tidal-power sets. The turbine has movable blades, and an examination of Fig. 12 will show that the Cambeyrac layout is not very different from that of Argentat. The hydraulic conduit is simi-

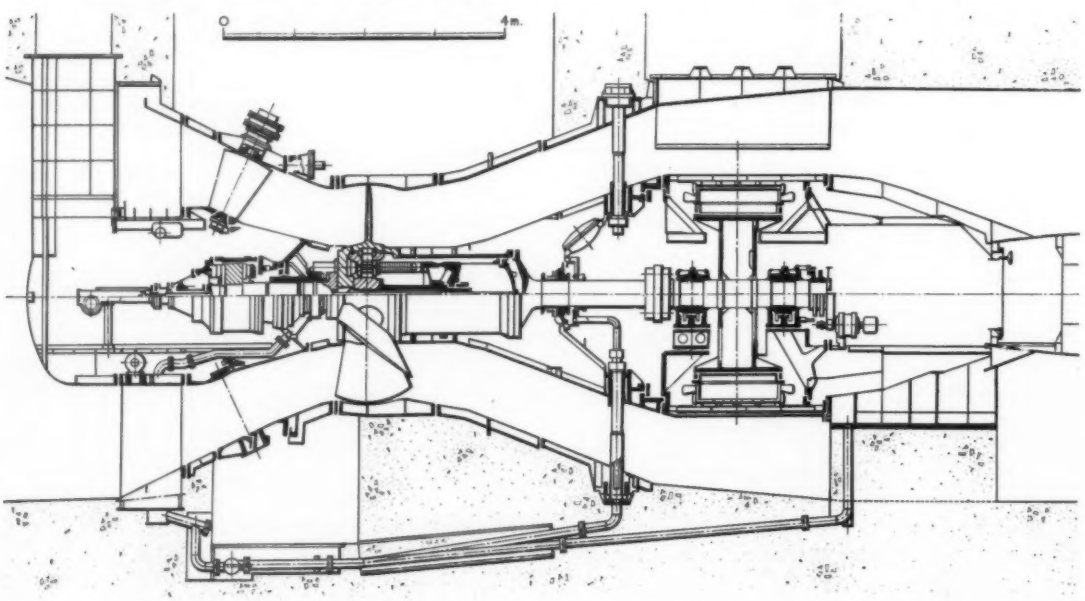

Fig. 12. Layout of the 5,000-kW downstream-bulb installation at Cambeyrac

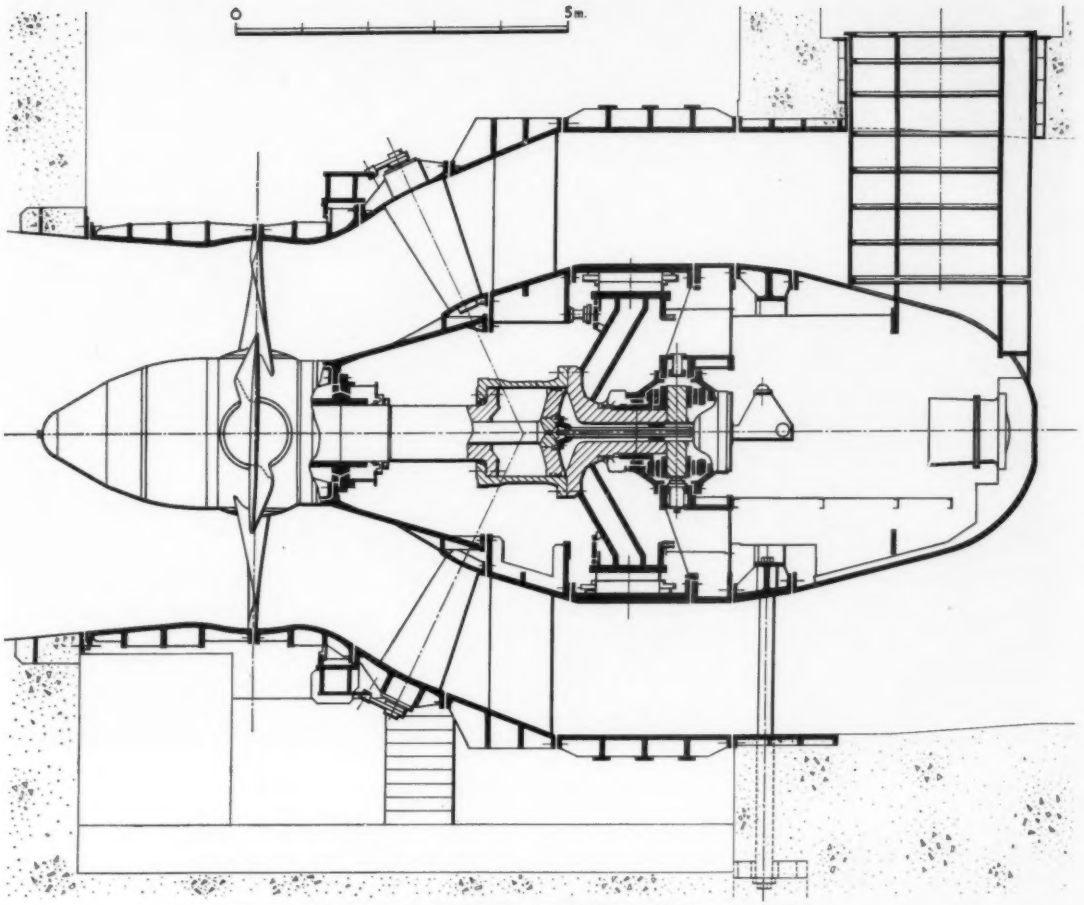

Fig. 13. The experimental 9,000-kW St. Malo installation

lar, and the order of the components, reading from upstream to downstream, is the same. We shall therefore confine ourselves to a description of those details which differ from those at Argentat.

The access shaft has a streamlined profile and terminates in a central dome. This shaft is not embedded in a concrete pier but is directly connected to the inlet cone, which is secured in concrete.

The movable guide vanes are similar to those at Argentat but have a completely different system of control. Each vane is moved by an independent servomotor, and synchronisation of movement between the vanes is secured by a crown wheel inside the bulb engaging with a bevel pinion keyed to the spindle of each vane. Safety devices prevent the crown wheel from being subjected to excessive forces. Risk of vane vibration during pumping in either direction of flow is prevented by a hydraulic locking device.

The thrust bearing, which is housed in the junction ring, is double-faced and is fitted with pads capable of tilting to accommodate rotation in either direction. In the thrust-bearing housing are the channels conveying oil to the runner servomotor.

The runner forms an integral part of the shaft assembly. To enable the blades to be reversed they are actuated by rack-and-pinion mechanism controlled by a servomotor in the runner hub. The extent of blade movement required explains the unusual length of the servomotor section of the hub.

The upstream alternator fairing, which carries a carbon gland similar to that located upstream of the runner, is secured to the surrounding concrete by six radial tie rods carefully prestressed during the mounting and adjustment of the bulb. The Alsthom alterna-

Fig. 14. Downstream ogive of Cambeyrac set

Fig. 15. Guide-vane and alternator housing for the St. Malo set

tor, which, with its shaft and two bearings, is a separate unit, is fixed on the one side to the fairing just described and on the other to the downstream fairing. This fairing, which forms a unit assembly, comprises the casing itself, a horizontal streamlined strut embedded in the concrete walls, and a vertical streamlined strut. The compartments of the horizontal strut are utilised for access to the alternator, for the passage of cables and piping, and for ventilation. The ventilating air is circulated by external fans through external coolers.

Later we shall discuss some of the numerous tests on this installation, which was placed in service in 1957.

St. Malo Upstream-Bulb Set

This is a prototype experimental set (Fig. 13) designed to operate as a turbine or as a pump in either direction of flow, and has movable runner blades and guide vanes. It is rated at 9,000 kW under a head of 5·50 m. The components of the set include the hydraulic conduit, the upstream hub, the thrust bearing and its supporting spider, the alternator, the inlet ring, the distributor, the turbine cone, the runner and the shaft.

The hydraulic conduit is very little different in construction from those already described, but except for the end rings it is not embedded in concrete. The set is therefore secured by supports bearing on the lower parts of the inlet ring and the alternator casing. The upstream ogive, the surface of which is used for alternator cooling, is removable and is located in the conduit by four prestressed tie rods placed on 45° axes. An upper streamlined rib affords access to the inside of the bulb and admits cables and piping.

The thrust bearing, which is carried in a supporting spider, includes a double-faced thrust ring on a spherical seating and tilting pads which can adjust themselves to either direction of rotation. The Alsthom alternator consists only of the stator and the rotor, the latter being welded directly to the upstream shaft and is thus integral with the runner hub.

The inlet ring carries the main weight of the set and in particular resists all horizontal thrusts exerted on the bulb. The distributor (Fig. 15) is identical in design with that of Argentat. All the guide vanes are controlled by a regulating ring actuated by two servomotors, and are locked hydraulically when the set is pumping.

The turbine cone carries the main bearing and the carbon gland. Access to the interior of the cone is obtainable through the thrust-bearing spider and between the arms of the alternator rotor.

The runner has non-reversible blades and is of normal design. It is overhung on the alternator shaft. The runner servomotor is located between the turbine shaft and the hub of the alternator rotor.

Many parts of the St. Malo set are made of stainless steel or are stainless-steel faced, and arrangements for cathodic protection are provided. The information to be gained by the operation of this experimental set will be of the utmost interest from the standpoint of the life in sea water of the various materials used, the mechanical functioning of the set, and its hydraulic behaviour.

The final article will consider problems of operation.

(To be continued)

Fig. 14. Crushing plant and aggregate stockpiles for Flaming Gorge project

The Upper Colorado River Project

This series of articles describes the constructional work on the Glen Canyon, Navajo and Flaming Gorge dams. This concluding article is devoted to Flaming Gorge dam

By IAN B. MACKINTOSH, M.A., A.M.I.C.E., M.ASCE.*

PART THREE

FLAMING GORGE dam, a unit in the US Bureau of Reclamation's Upper Colorado River project, is located in a rugged canyon (see Fig. 15) on the Green River in a remote part of north-east Utah. The contract for the construction of the dam was awarded in June 1958 to a joint venture which is made up of Peter Kiewit Sons Co. (sponsor), Morrison-Knudsen Company and Brown & Root Inc. for a contract sum of $29,600,000, some 12% above the Bureau of Reclamation's estimate for the cost of the work.

Flaming Gorge Dam

The dam will be the seventh highest concrete dam in the USA, rising 490 ft above bedrock. It is a thin-arch design with a maximum thickness of 150 ft tapering to 27 ft. The crest length is 1,180 ft and the crest elevation 6,047 ft. The quantity of concrete is 1,000,000 cu yards. The dam will create a 91-mile-long reservoir having an active capacity of 3,800,000 acre ft. and, in addition to its primary feature as a regulating unit, the dam will have a 108-MW power plant. The latter will be located at the foot of the dam and will receive its water from three 10-ft-diameter penstocks embedded in the heart of the dam. The spillway is formed by a tunnel inclined at 55° to the horizontal and tapering from 26·5 ft diameter at inlet to 18·0 ft diameter at outlet. The spillway tunnel is approached by a high-level channel cut in the left abutment, and is controlled by fixed-wheel vertical-lift gates. The peak flood recorded was 21,000 cusecs and the annual run-off from the 7,670 sq mile drainage area is from 1·5 to 2·0 million acre ft.

In the dam the temperature of the concrete must be below 80°F when placed. (It is interesting to compare this with below 50°F, which is the requirement for concrete in Glen Canyon dam.) The specifications call for the embedding of a 1-in diameter cooling-water pipe in the bottom of each lift and for the circulation of river water through these pipes. The maximum temperature reached by the river water is 72°F. The grouting of the construction joints is carried out during the winter, and the dam is grouted in 60-ft grouting lifts.

Recording instruments provided in the dam are as follows: Electrical resistance strain meter, thermometer wells in the galleries for inserting thermometers.

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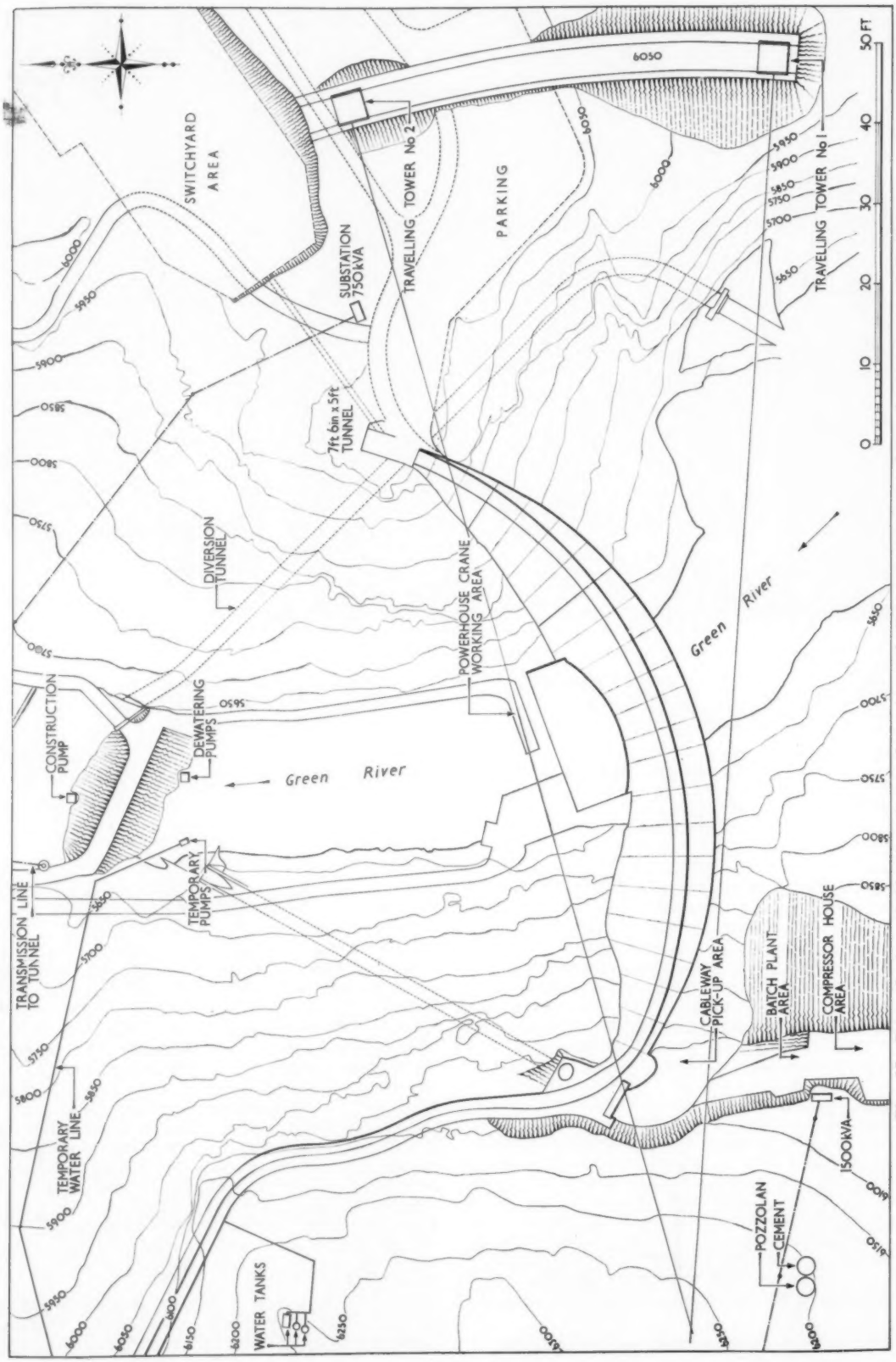

Fig. 15. General plan of Flaming Gorge dam site including temporary works and constructional facilities

Fig. 16. Placing concrete in the dam by means of an 8-cu-yard bucket

remotely recording thermometers, joint meters to show opening of contraction joints, deflection targets on the downstream face, and five 16-in-diameter plumbing shafts extending down through the dam to the lowest gallery.

The dam is formed into blocks for construction purposes and these are about 50 ft in length (see Fig. 16). The contractor is given the option of using a 5 ft lift height or a 7 ft 6 in lift height, and he has chosen to use the latter. The higher lift reduces the amount of clean-up and preparation of horizontal construction joints, and at the same time reduces the total number of pours in the work. On the other hand the forms have to be more robust to carry the heavier load with a higher lift, and the system also gives a smaller number of uses for each form. However, it is considered that 7 ft 6 in is about the limit to which the contractor would wish to go for economic reasons. The forms are of the steel-truss type and are being supplied by the Dixey Form Company, of San Antonio, Texas.

All forms on the dam may be stripped 24 hours after concrete has been placed against them, but no lift of concrete may be placed on top of a previous one which has been placed for less than 72 hours. This is to allow the maximum rate of loss of heat of hydration immediately after placing. In addition, the specifications call for a maximum difference in elevation between adjacent blocks of 30 ft and a maximum difference over the entire length of the dam of 52 ft. These limits apply to 7 ft 6 in lifts, the limits for 5 ft lifts being lower.

Foundation Grouting

The Bureau have developed a fairly standard procedure for foundation grouting. A programme of grouting "B" holes 1½-in diameter to a depth of 30 ft and a spacing of 20 ft centres is carried out over the whole area of the base of the dam, using medium pressures of about 100 lb/in². This is to fill any cracks in the surface of the foundation and to solidify the substrata. The foregoing is carried out before any concrete has been placed. The second stage of grouting is carried out from the grouting and drainage gallery, which follows the general profile of the dam's foundation in a series of steps. The grouting holes in the gallery are the "A" holes, and are formed by 2-in-diameter pipe left embedded in the concrete and inclined at an angle of 10° to the vertical; the "A" holes are on the upstream side of the gallery spaced at 5 ft centres. On the downstream side of the gallery 2-in-diameter pipes are embedded reaching down to the foundation rock, and these are fanned out from the grouting holes also at 10° to the vertical for drainage to prevent any water that gets past the grout curtain from establishing uplift forces under the dam. It is usually only necessary to grout the "A" holes at 10 ft centres. No "A" hole may be grouted until

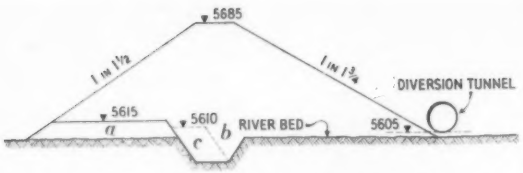

Fig. 17. Cross section of upstream cofferdam

the concrete within 200 ft of the hole has been placed to a height of 100 ft above the foundation and the contraction joints within the area grouted to a height of 60 ft. The "A" holes are 200 ft or so in length and use about 1 lb of cement per foot of hole. The "A" holes are grouted at a high pressure of about 250 lb/in².

In the winter time at Flaming Gorge the temperature reaches 24° below zero, and during the first winter work was carried out on excavation. Concreting work is shut down during the winter, which lasts from late November to early April. The concreting programme is as follows: 50,000 cu yards in 1960, 500,000 cu yards in 1961, and 450,000 cu yards in 1962.

Diversion Tunnel

The river was diverted by means of a 1,200-ft-long concrete-lined tunnel, which has a finished diameter of 23 ft and a carrying capacity of 18,000 cusecs. Although this tunnel is only a temporary feature of the work and will be plugged eventually, the dimensions and specifications are laid down by the Bureau. Tunnel driving started in early January 1959, and was completed by May 1. Lining was then commenced and was completed by mid-August 1959 (see Fig. 18). The lining operation progressed on a five-day and three-shift basis: moving the form on swing shift, cleaning up on graveyard shift, and concreting on day shift. Various minor troubles were encountered in this operation, and the most critical was getting the concrete mix right so that the concrete could be successfully pumped across the river and up the tunnel to the forms; nevertheless there were two weeks when five lengths of lining were placed. The lining was placed in 30 ft lengths in the straight portions and in 20 ft lengths on the two bends. The design thickness of the lining was 1 ft but overbreak caused the average thickness used to be about 2 ft. A section has been left in the diversion tunnel to take a solid concrete plug 60 ft long, and in this section the concrete lining tapers from 29-ft diameter back to the 23-ft diameter of the remainder of the tunnel.

Cofferdams

The upstream cofferdam has a crest length of 600 ft and a volume of 200,000 cu yards. A cross section of the cofferdam is shown in Fig. 17 and the foundation procedure was as follows:—

1. Construct rockfill dyke *a* by end-dumping.
2. Excavate cut-off trench *b* from dyke using a dragline.
3. Place sandy-silt mat *c* (12,000 cu yards) to make underwater cut-off.
4. Complete remainder of cofferdam using rock excavation from the dam, selected to give a high shale content, the material being end-dumped and then spread with a D.8 bulldozer.

Boulders were encountered in the excavation of the cut-off trench for the cofferdam, even though the cores had shown a tight bottom. As a result the cut-off trench was not taken down to bedrock. The seepage through the cofferdam was nevertheless quite low, and it is estimated that during the 1960 spring flood the inflow was about 250 g.p.m., which was comfortably dealt with by a 10-in horizontal Deming pump.

The downstream cofferdam was 250 ft long, had a crest elevation of 5,620 ft, and contained 20,000 cu yards of material.

Keyway Excavation

On this dam the excavation of the abutment walls is a tricky operation. The walls are very steep, and on the right abutment in particular work is made more difficult by the awkward way in which the rock breaks up. The ideal would be to form a flat bench on which men can work with jack-hammers and a bulldozer, the material being pushed over the edge

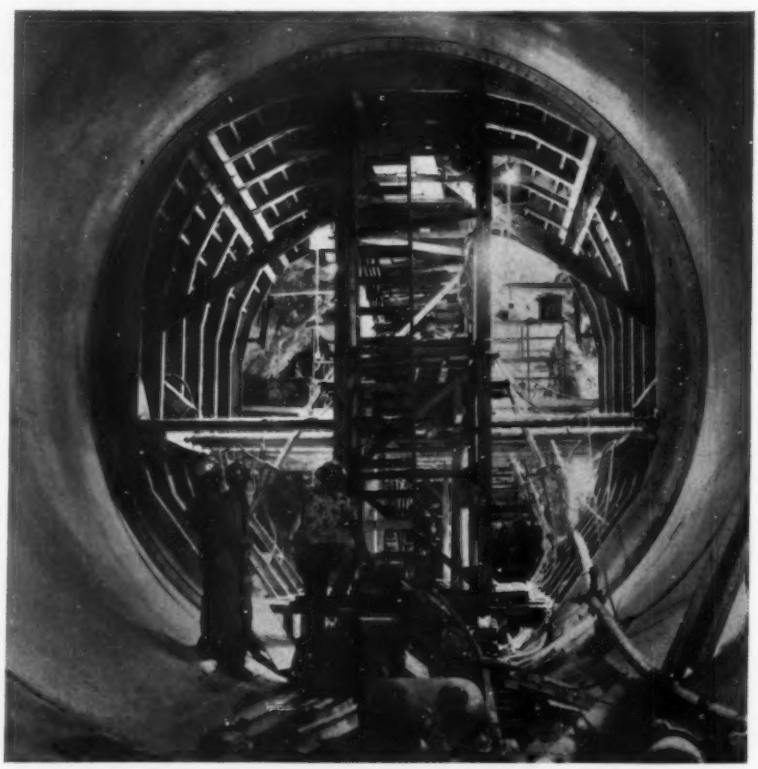

Fig. 18. Forms for diversion-tunnel lining in position

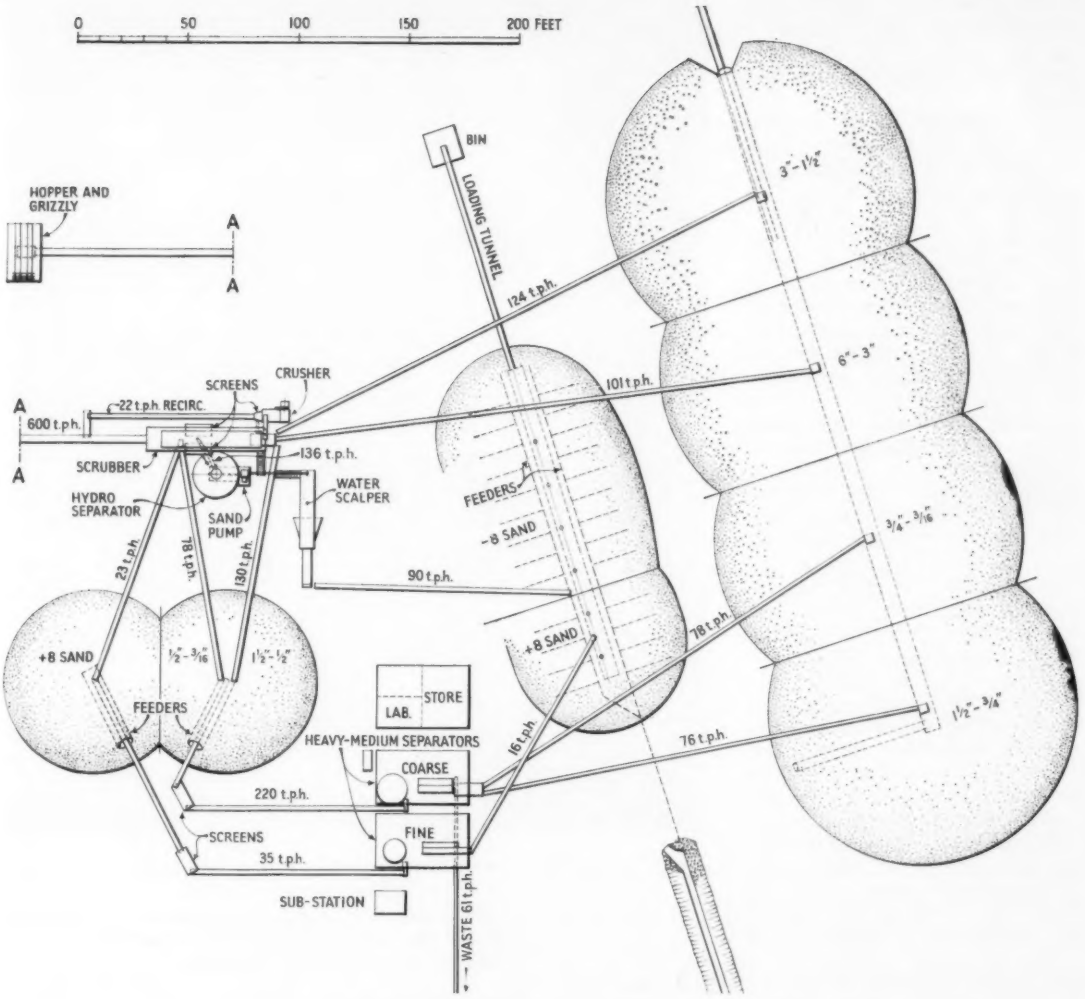

Fig. 19. Plan of the aggregates processing plant

into the canyon. If the work is straightforward a depth of 50 ft of abutment should be completed in a month, the time taken being governed by the time to hand trim the foundation; where the foundation is broader more men can be employed on the work. In the right abutment there are 25,000 cu yards of excavation and the unit cost of this work is about $5·50 per cu yard.

Aggregate Plant

Materials for concrete aggregate are excavated and crushed at a gravel deposit 15 miles from the dam site. The gravel is hauled by three Caterpillar DW 21 scrapers, which are assisted in loading by a Caterpillar D.9 pusher, to a grizzly which scallops off the +6 in size. At the crushing plant (see Fig. 14) the material is screened into 3–6 in and 1½–3 in sizes and placed in finished stockpiles. The under-No.-8 size passes through a classifier and thence to finished stockpile. The sizes between No. 8 and 1½ in are then subjected to a heavy-medium separation treatment to exclude any stone having a specific gravity of less than 2·50, the aggregate being said to have been "beneficiated" after that treatment. The techniques used to exclude these lighter materials are well known in the mining of ores, but this and Glen Canyon dam are the first occasions when heavy-medium separation has been used in the construction industry. The materials are handled in two streams, the No. 8 to No. 4 size in one and the No. 4 to 1½ in sizes in the other. In each stream the principal piece of equipment is a cone which contains a liquid of ferrosilicon of 2·50 specific gravity, and when the materials are suspended in this the heavier material passes down through the liquid and the light material is floated off at the surface.

In Fig. 19 is shown a plan of the concrete-aggregates processing plant, which has an average capacity of 220 tons per hour. The finished materials are in two lines, one for sand and the other for coarse aggregate. Under the sand line a 24-in conveyor in a 7-ft diameter reclaim tunnel formed by liner plates is used to feed a 60-ton loading hopper. Similarly under the coarse-aggregate line a 30-in conveyor belt is used in an 8-ft-diameter tunnel to feed a 240-ton hopper, which is subdivided into four compartments. From here the materials are hauled in

bottom-dump trucks the 15 miles to the dam site, where the materials are washed and rescreened before passing into the concrete mixer.

The aggregate plant includes 2,878 linear feet of conveyor belt, mostly in the 24-in size, which has a normal speed of 200 ft/min. The absolute maximum inclination used on the conveyors is 18° to the horizontal. The plant has a total connected horsepower of 1,469 and a maximum demand of 1,080 h.p., and requires 5,300 US g.p.m. of water and 710 ft^3/min of compressed air to operate it.

Batching Plant and Cableways

The concrete for the dam is batched and mixed in a Noble automatic plant located on the left bank, comprising four 4-cu-yard tilt-type mixers. The concrete is discharged into a wet hopper under which passes a transfer car mounted on rail track, which carries to side-discharge hoppers each of 8-cu-yard capacity. (See Fig. 20.)

Concrete is placed in the dam by means of 8-cu-yard bottom-opening Blaw-Knox buckets, the gates of which are powered by automatically recharged hydraulic rams. The gates are set in motion by a man standing near the point of discharge of the bucket, and the hydraulic system avoids the loss of time coupling up to the bucket a compressed-air hose as in the widely used Garbro buckets.

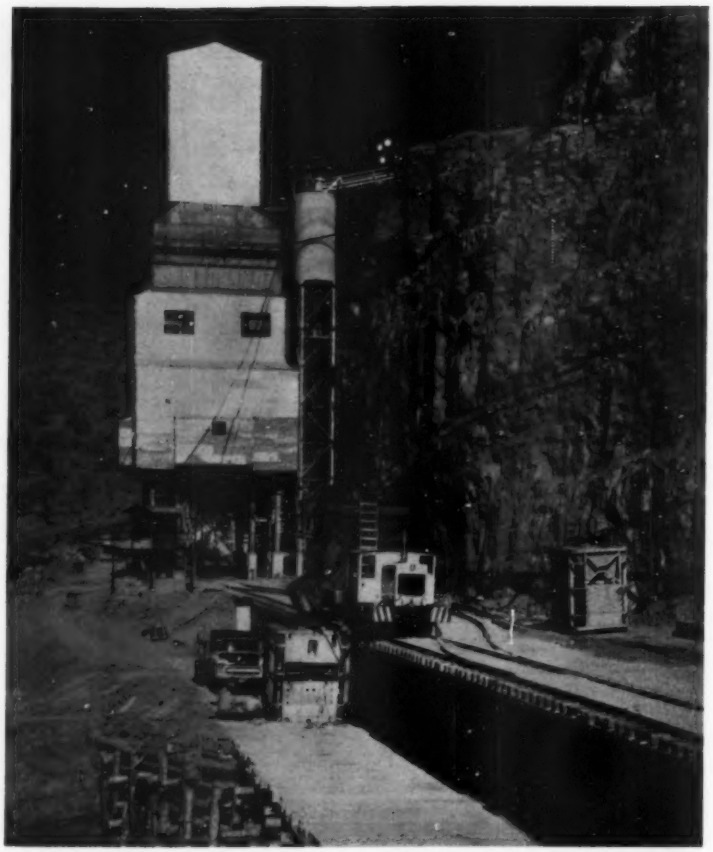

Fig. 20. Batching plant with concrete-transfer car on track in foreground

The 8-cu-yard buckets will be handled by two 25-ton cableways which have a common head tower located on the left bank. The 75-ft-high tail towers travel on a common radial track formed on the right bank by means of a moderate amount of cut and fill. The length of the tail-tower track is 750 ft, and the arrangement is such that all parts of the dam can be reached by the cableways. The cableways are the standard American type and are made by Lidgerwood. In this type the load line has four parts under the carriage and is dead-ended on the carriage. The span of the main track cable is 1,900 ft, and twin button lines will be used to space out the slack carriers as the carriage runs out from the head tower. The concrete pick-up point is on the same side of the canyon as the fixed head tower, and so the transfer car has only a short travel to the point where the cableway is operating. The hopper on the transfer car is discharged directly into the buckets, so that the bucket does not have to be unhooked from the cableway, thereby saving time. The tail towers came from the Travelift cableways used at Detroit dam, but all other cableway components are new.

Programme

The work started in August 1958, and by October 1959 the diversion tunnel was driven and lined and the keyway excavation proceeding down the abutments. Concreting in the dam started in August 1960 and will be completed by the end of the 1962 season, according to the present schedule.

Acknowledgments

The author wishes to extend his thanks to Mr. Douglas Baker, Project Manager, and Mr. Henry Scott, Project Engineer, of Peter Kiewit Sons & Company, and to Mr. Jean Walton, Project Construction Engineer, US Bureau of Reclamation, for their kind co-operation during his visit. Acknowledgment is also made to the Bureau of Reclamation for permission to use the photographs reproduced in Figs. 14, 16, and 20, and to *The Em Kayan* for Fig. 18.

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Paulo Afonso Turbines. In our November 1960 issue we published an abstract of an article that appeared in the July 1960 issue of *Die Wasserwirtschaft* on the Rio Sao Francisco development. The two first-stage turbines in the Paulo Afonso station were built by Dominion Engineering Company, but in the original article these machines were wrongly credited to another firm, and this incorrect information was reproduced in our abstract. A correction note has now appeared in *Die Wasserwirtschaft*.

Remedial Works at Bhakra Dam

This article describes the sequence of events at the time of the failure of the hoist chamber in the right bank, and gives an account of the remedial measures that were carried out

By RICHARD J. SALTER, B.Sc.(Eng.)(Hons.), A.M.I.C.E., A.M.I.Mun.E.

THE Bhakra project, which forms part of the programme to provide more electricity and irrigation water for India, has attracted the attention of engineers throughout the world. The most impressive engineering work in the project is the massive Bhakra dam, which contains and regulates the flow of the River Sutlej. The dam, which is of mass-concrete construction, has a maximum height of 741 ft, and was the scene of one of the largest engineering disasters of recent times. This occurred when, on August 21, 1959, water passing through the right-bank diversion tunnel broke through into the dam galleries, and caused considerable damage and loss of life.

To allow construction of the dam to proceed, the river flow had been diverted during the early stages of the work through two 50-ft diameter concrete-lined tunnels, half a mile in length, which were driven through the abutments. Before the 1958 monsoon season the left-bank diversion tunnel was plugged with concrete. The right-bank tunnel, however, was blanked off with an initial concrete plug only, which had two 11-ft by 22-ft high openings left in it. Each opening was provided with two hydraulically operated gates, one for emergency use and the other for flow regulation. The hoisting mechanism for these gates, each of which weighed 70 tons, was built into a hoist chamber constructed in the rock above the gates, at a depth of some 300 ft below the crest of the right abutment.

The hoist chamber was lined with reinforced concrete. It has a length of 42 ft, a width of 30 ft, and a height of 57 ft. The primary purpose of the regulating gates in the conduit beneath the chamber, was to store water in the reservoir during the construction period, and to control the amount of water released from the reservoir. They were also expected to assist in the final plugging of the conduit when the lower river outlets were completed in the dam itself. Satisfactory control of irrigation water was possible, and during 1958 a half a million acre ft of stored water was released for irrigation purposes. During 1959 three times this amount of water was released.

In the monsoon period of 1959 excess water was passed over the centre of the dam, which was at a lower level than the abutments, and also through the right-bank conduit. The first sign of the impending disaster was seen in the afternoon of August 21, 1959. Small hair cracks appeared in the floor of the hoist chamber, slight separation of a steel column occurred, and a small amount of water flowed into the adit tunnel. At this time an estimated flow of 25,000 cusecs was passing through the conduit.

By 4 p.m. nothing further had occurred but, as a precaution, the river outlets through the dam were opened, and an attempt was made to close the emergency gates of the hoist chamber. This was unsuccessful as the gates stuck when only 8 in from the bottom seating. The regulating gates were then lowered to give increased relief to the dam, and construction workers were ordered to leave the dam as quickly as possible. One hour later the amount of water entering

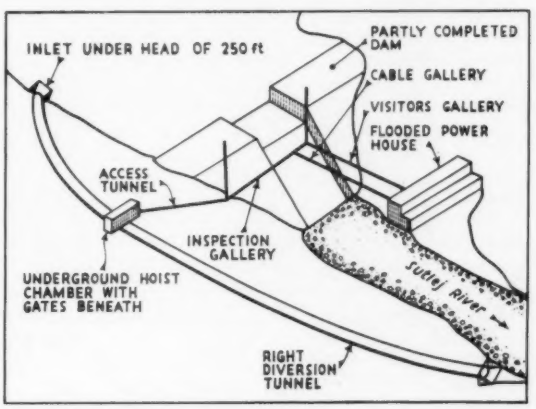


Fig. 1. Sketch showing the arrangement of the workings when the disaster occurred

the hoist chamber and the adit had increased, and sounds of rupturing metal were heard. At 6 p.m. the hoist chamber collapsed, and the foreman and nine workmen still remaining in the chamber were killed. The inflowing water filled the galleries within the dam, and also the power house.

The disaster received a great deal of adverse publicity in India where hopes of a substantial rise in the standard of living were focused on the construction of this and similar irrigation and power projects. It was thus essential to begin remedial works immediately. The first problem was the dewatering of the left-bank power house so that damage to the installed equipment could be limited and repair work begun. It was, of course, also essential to plug the water flow in both the right tunnel and the hoist chamber, where the water was gaining access to the interior of the dam under a head of 250 ft.

At the time of the failure, five 90-MW units were already installed in the power house, and each one had a triple draft tube closed by gates and temporary walls. Water flowed into the power house through the access and cable galleries, and had already submerged three generators when one of the draft-tube gates collapsed under the water pressure, and so prevented the water rising any higher.

Two openings were blasted in the cable gallery

to divert water entering the power house through the gallery, but only a small reduction in flow took place. A second attempt to arrest the flow from this source was then made by sinking a 30-ft deep shaft into the top of the gallery. A 25-ton gate was then lowered down this shaft, but it was washed away. Guides were then fitted into the shaft and a 40-ton gate was lowered into the cable gallery. This attempt was successful, and only a small amount of water then leaked into the power house from this source. Water entering the power house from the access gallery was then diverted by means of a flume, which was heavily reinforced with steel girders. Finally, the displaced draft-tube gate, which had been recovered from a depth of 53 ft by divers from the Indian Navy, was replaced. Work then began on pumping out the power house, using pumps with a capacity of 45 cusecs, and these were later replaced by 75-cusec capacity machines. The power house was pumped dry just over two months after the failure in the hoist chamber.

The problem of repairing the damage to the hoist chamber and the right-bank tunnel was a more vexed one, and a five-point plan was decided upon after several expert opinions had been obtained. It was decided that an attempt would first be made to choke the upstream portal of the right-bank tunnel by dumping crates and precast concrete blocks, followed by a filter of graded aggregate, sand and clay, which would seal up any space between the larger particles of material.

The second blocking operation would be to sink a 60-ft deep by 12-ft diameter shaft into the tunnel at a point about 100 ft upstream of the hoist chamber. It was hoped to assist the tunnel blocking operation from this point.

Thirdly, it was planned to drive a 300-ft long 8-ft square approach tunnel to reach the top of the hoist chamber from the existing railway tunnel.

Another tunnel was planned to be driven from the existing railway tunnel to the initial plug in the right-bank conduit. This tunnel would have a length of 850 ft, and a cross section measuring 12 ft by 15 ft.

The fifth measure proposed was to construct a 350-ft long bypass tunnel, 10 ft by 14 ft in cross section, to connect the two sections of the diversion tunnel on either side of the initial plug. If the first two measures failed to result in the dewatering of the hoist chamber then it was proposed to use this tunnel to divert the flow away from the hoist chamber. After further consideration the fifth measure was abandoned because of the danger of blasting so close to the dam. A further difficulty was estimating what would happen when the upstream end of this tunnel finally connected with the right-bank conduit upstream of the initial plug, where the water was under a head of 300 ft.

The upstream portal, which it was planned to block, had a net effective opening of 932 ft², and, under a 300-ft head of water, this intake was able to pass over 100,000 cusecs.

It took six weeks for the reservoir level to be lowered sufficiently, and when work commenced on October 10, 1959, there still remained 275 ft of water above the tunnel portal. A grid was set out over the area where dumping was to take place, using steel-wire rope stretched across the river. The centre of the upstream portal was then pinpointed by a pendulum hanging from the steel ropes, and 50-ft downstream of the tunnel portal, and on the line of the tunnel, a mooring raft was anchored. The whole area was illuminated by floodlights so that night work would be possible.

Crates manufactured from Sommerfield track, which consisted of 10-ft lengths of ¼-in-diameter steel bars welded onto a steel wire mesh, were the first to be dumped. The crates measured 5 ft by 5 ft by 6 ft and 10 ft by 5 ft by 6 ft, and weighed 10 tons and 20 tons respectively when filled with concrete. The crates were lined with several layers of sacking, and a rich concrete mix was then poured into them. The crates were then placed within an hour of pouring, so that firm bedding of the plastic blocks would be achieved. The blocks were dumped from barges of 40-ton capacity, and the dumping was in accordance with a strict pattern, the execution of which was facilitated by the grids.

Precast concrete blocks weighing between 10 and 20 tons were then dumped. At intervals crates were dropped to provide a cushioning effect, and to reduce excessive breakage of the precast blocks. These blocks were of many different shapes, such as cubes, tetrahedrons and rectangular blocks, so that a better interlocking effect would be obtained. It was intended to reduce the flow through the portal considerably by using the plastic crates and the blocks, although there was a considerable risk that they would be swept down into the conduit by the greatly increased velocities as closure was neared. This did in fact take place, and it was only by persistent and fast dumping of the precast blocks that, on November 22, 1959, a considerable reduction in flow was observed.

It was next necessary to fill the voids between the blocks, and cover the portal area with impervious material. For this purpose small crates and blocks of 1 cu yard size were dumped, followed by a 3-ft thick layer of large rock. After this, a 12-ft thick layer of 7-in gauge stone, a 3-ft thickness of 3-in gauge stone, and a 2-ft thickness of 1½-in gauge stone were spread evenly over the portal area. These measures reduced the flow from 9,000 cusecs to 3,700 cusecs. By dumping gravel and sand on top of the stone blocks the flow through the conduit was further reduced to 1,200 cusecs. On January 3 dumping of the clay seal was commenced and, by January 25, the leakage was reduced to 374 cusecs. Further loads of clay were dropped to reduce any future leakage, and by the completion of the operation, a total of 520,000 cu yards of clay had been placed.

In the early part of February, 1960, work was started in the damaged hoist chamber. Six syphons of 2-ft diameter and one of 1-ft diameter were installed, thus making it possible to dry out the chamber, and plug it completely. Long concrete plugs were cast in the right-bank diversion tunnel, and remedial work in the remainder of the dam was completed before the outset of the 1960 monsoon.

Trent Valves. A comprehensive catalogue covering a large range of valves for small-bore pipes has been received from The Trent Valve Co. Ltd., 47, Great Eastern Street, London, E.C.2, from whom copies may be obtained.

NCK-Rapier 405. This excavator is fully illustrated in a pamphlet (publication no. 405/3) by NCK-Rapier Limited, 32, Victoria Street, London, S.W.1. It has a bucket capacity of 1 cu yard when employed as a face shovel.

Hydraulic-Ripping Technique at Maraetai

Fig. 1. In the foreground a Euclid TS24 scraper loads ripped rock with the assistance of an International TD24 dozer. In the background a Caterpillar D9 and two International TD24s, all equipped with hydraulically controlled ripper units, are ripping the rock in readiness for loading

Hydraulic rippers attached to tractors were first used in New Zealand a few years ago during excavation work on the site of Wellington Airport. Since then they have been extensively employed by the Ministry of Works on the Benmore and Maraetai II hydro-electric projects.

Rippers have been used on the excavation of the intake channel at Maraetai II since November 1959 and the results have been most satisfactory. Large quantities of rock have been removed for less than half the unit cost ten years ago using explosives, trucks and shovels. The monthly average quantity of excavation was 200,000 cu yards and the maximum figure of 250,000 cu yards was probably four times that achieved on previous works of this type in New Zealand. The Maraetai II intake channel has an approximate volume of 1,300,000 cu yards, and most of this quantity was removed in six months.

The rock at Maraetai is ignimbrite, a massive igneous material that breaks into small pieces when ripped, and the hardness of which varies considerably. The top sheet can be dug with a pick and ripped without difficulty, and it was broken up at a rate of over 200 cu yards per hour. Middle-sheet ignimbrite, through which most of the Maraetai II intake channel has been dug and in which ripping was mostly carried out, is harder. Lower-sheet ignimbrite, in which the foundations for the dam are being established, is harder still, and blasting was necessary. Ripping of lower-sheet ignimbrite became so difficult—the output decreased to 80 cu yards per hour—that costs exceeded those for blasting and shovel loading, and the method was abandoned. As the illustrations accom-

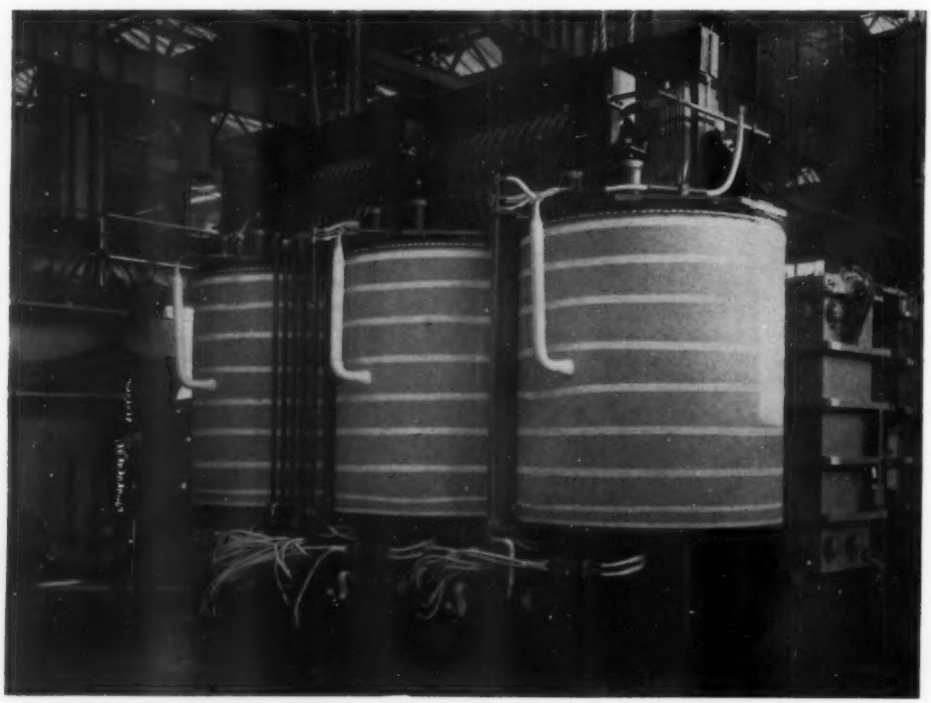
Fig. 2. Maraetai II intake channel. The far face has been trimmed with a Caterpillar D9 fitted with an angle blade

panying this article show, the walls of the intake channel have been neatly trimmed, although no hand work was necessary. A Caterpillar D9 tractor, fitted with an angle blade, trimmed the batter as excavation proceeded by using the point of the blade as a plough. No other machines available on the Maraetai II project were heavy enough for this job and a D8 tractor merely bounced off.

All material was removed by scrapers. At first crawler tractors were used but as the haul increased rubber-tyred tractors replaced them. A variety of equipment was used, including International TD24s and 25s, Euclid TS24s and 18s, and Caterpillar D8s and D9s. Most of the ripping was done by D9s. A feature of the excavation work at Maraetai was the good results obtained by using the heaviest-push bulldozers and tractors to assist in loading scrapers. Even the output of double-ended Euclid TS24s was doubled by push loading with Caterpillar D9s.

To achieve success in ripping it is necessary to use the heaviest and most powerful tractors. On the recently started Aratiatia project tandem ripping with two D9s will be attempted when the power-house foundations are excavated. In this way it is hoped to avoid drilling and blasting although it must be emphasised that ripping has certain definite limitations and is very onerous work. Teeth wear away quickly and in lower-sheet ignimbrite they last only three hours. Under certain conditions costs may be as high as for drilling and blasting.

The Robert Moses Niagara Transformers



The transformers for the recently commissioned Robert Moses power station, part of the Niagara power project (a description of the civil-engineering works was given in WATER POWER, August 1960), were supplied by Ferranti Limited, of Hollinwood, Lancashire, England. The contract, which was valued at $3,913,770, included the supply of six 180 MVA, 13·2/120 kV, 3 ph, 60 c.p.s. generator transformers and seven 180 MVA, 13·2/240 kV, 3 ph, 60 c.p.s. generator transformers.

The station is equipped with 13 hydraulic turbines, each rated at 200,000 h.p. at a net head of 300 ft, and which are directly connected to 13 generators, each rated at 167 MVA and operating at 0·90 p.f., 120 r.p.m., 3 ph, 60 c.p.s. Each generator has an associated transformer rated at 180 MVA. Seven transformers will be used to supply the 120-kV section of the switchyard, and six to supply the 240-kV section. The connection between any generator and its transformer will be by isolated phase bus, and the connection between the HV terminal of any transformer and its corresponding bay in the switchyard will be made by oil-filled cable.

The transformers are of the forced-oil, forced-water-cooled, inert-gas-filled type and the windings have graded insulation for operation on a solidly earthed system, and incorporate directed oil flow. Two oil-to-water heat exchangers are mounted on each transformer, each heat exchanger being suitable for dissipating the transformer losses corresponding to 70% of the rated full-load capacity. Each transformer is fitted with an external manually operated off-load tap changer on the HV windings.

The tanks and covers are of welded-steel construc-

tion capable of withstanding an internal gas pressure of 10 lb/in² at the top of a tank when filled with oil to normal level. The tanks are also capable of withstanding full vacuum. Fully automatic pressure-regulated inert-gas equipment is provided for maintaining an adequate supply of oil-pumped dry nitrogen gas under positive internal pressure in the space between the top of the oil and the tank cover.

A full series of routine and type tests in accordance with American Standards C 57.11, C 57.12 and C 57.22, and NEMA Standard TRI was carried out on each transformer prior to its despatch from the Hollinwood factory. These included loss tests, full-load-temperature tests, noise tests and impulse tests at insulation levels of 450 kV and 825 kV on the 120-kV and 240-kV windings, respectively.

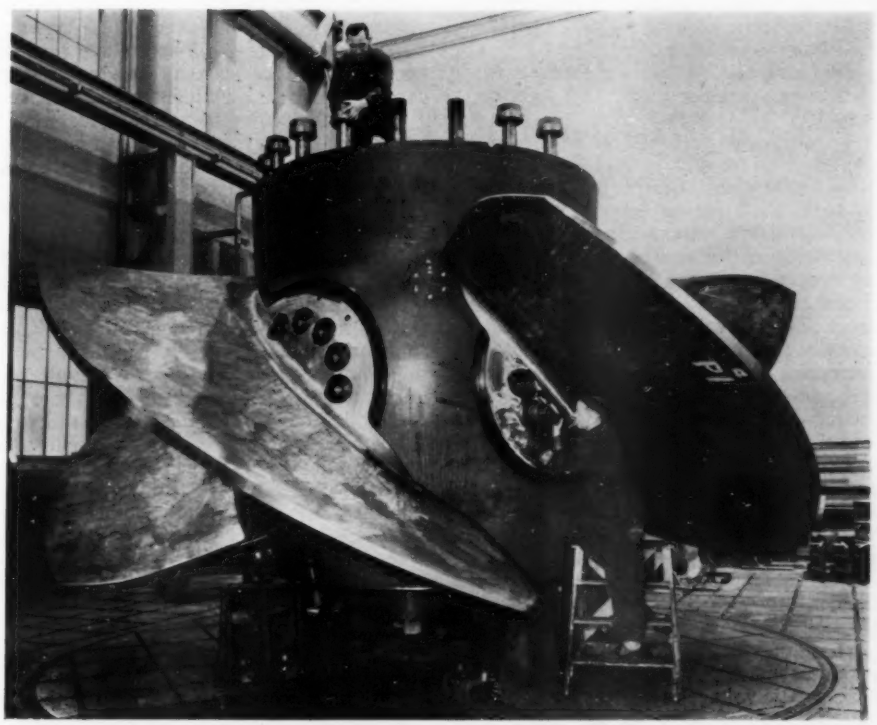
Each 13·2/120 kV transformer weighs 154 tons, and the 13·2/240 kV units weigh 175 tons apiece. The smaller transformers have the following overall dimensions: height, 21 ft; width, 14 ft; length, 23 ft.; The larger units have the following dimensions: height, 23 ft 6 in; width, 14 ft 9 in; length, 23 ft 3 in.

The consulting engineers for the project are Messrs. Uhl, Hall and Rich, of Boston, Massachusetts, and the British inspection and progress engineers are Messrs. Merz and McLellan. An article on the station will appear in a future issue.

Accident-Prevention Device

In 1959, in the United Kingdom alone, there were 25,000 notified accidents from falls, of which over 200 were fatal. To provide a safeguard against such accidents, Neldco Processes Limited, Crossway House, Bracknell, Berkshire, have devised the Sala Safety Block, which is a spring-loaded safety line with automatic braking. It has been awarded the first certificate of approval issued by the Accident Prevention Division of the Midland Employers' Mutual Assurance Limited, and has been tested by a member of the manufacturer's staff, wearing the appliance, jumping without mishap from the cross arm of an electrical transmission tower.

Turbines for Porsi Power Station



The accompanying illustration shows one of the two runners being assembled for the Porsi power station on the Lule River in northern Sweden. The turbines, which are being manufactured by Nydqvist & Holm Aktiebolag, of Trollhättan, Sweden, will each develop 118,500 h.p. under a head of 33 m, and will operate at a speed of 115 r.p.m. It is claimed that they will be the most powerful Kaplan turbines in Sweden, and among the most powerful units of their type in Europe.

The Swedish State Power Board ordered two machines for the first-stage development of the Porsi station, and provision has been made for the eventual installation of a third set. The two turbines at present being manufactured are expected to be delivered this year.

Book Reviews

Stabilité de Réglage des Installations Hydroélectriques. By Prof. Lucien Borel, l'Université de Lausanne. Published by Editions Payot, Lausanne, and Dunod Editeur, Paris. 233 pp., 81 ff. Paper-board binding. Price 26 Sw. Fr.

The overall regulating stability of hydro-electric installations is a complex problem in which approximate solutions involve many unsatisfactory simplifications and complicated calculations, although mathematical techniques and computers now allow them to be treated without disregarding fundamental considerations. Borel's new general study, however, covers all the system effects that intervene, including precise turbine and governor behaviour—factors neglected hitherto. It is, however, limited to primary regulation but may be extended to secondary regulation later.

The first part is devoted to the theory of regulation phenomena and their mathematical resolution, and the second to numerical calculations made in the same order. This commendable example should be followed in other fields involving complex and interrelated effects, since it offers an appreciation of the actual values concerned.

Notation and non-dimensional considerations are followed by two chapters on turbine characteristics expressed in dimensionless head, discharge, efficiency and guide-vane-opening parameters on the contour diagram, from which four basic turbine hydraulic parameters are derived for any operating point, without restricting assumptions, except that departures from steady-state performance can be linearised. Seven other interrelated parameters can be calculated from them. Since turbine performance curves and surfaces cannot be expressed analytically these parameters must be calculated numerically from them when available. Typical values, however, are established statistically for maximum efficiency and output, and presented graphically as a function of a specific-speed parameter. This discloses inadmissible errors in assuming that turbines behave as orifices, which are merely special cases of Borel's analysis.

The basic study relates to isolated systems for which system-component equations are easily and logically deduced for the intake tunnel, surge-chamber junction, velocity head, surge chamber proper, pipeline, head losses, net head, system mass oscillation, waterhammer and other appropriate characteristics, including turbine efficiency, discharge, output, and speed relationships.

A clear exposition of generator and load characteristics introduces machine and system mechanical inertia, and automatic voltage regulators with voltage/frequency statism. Since the latter are faster than speed regulators the influence of system load on voltage is neglected.

The section on speed regulators contains a practical mathematical analysis of accelerometer and tachometric governors. The typical diagrams illustrating the main devices and their characteristics remove much of the mystery from component and overall performance. The analysis includes equations for: (a) The distribution valve in terms of servomotor stroke, speed-variation-device settings, accelerometric adjustment and servomotor/speed statism; and (b) governors with temporary speed adjustment with dashpot and temporary and permanent statism; and concludes by showing that both governor equations have the same form with accelerometric adjustment corresponding to dashpot relaxation time and tachometer/servomotor stroke/speed/time characteristics to temporary statism.

The fundamental equations for the primary waterways, surge chamber, turbine, governor, alternator, voltage regulator and load, are summarised in an approach to transfer functions and schematic block diagrams covering the whole installation. These are simplified by systematic reduction, and because they result from a rigorous system analysis and embody all the intervening characteristics, prepare the groundwork for possible installation improvements. The four overall transfer functions can be adapted to any external disturbance using open and closed loops for forced and free oscillations. For example the behaviour of an installation without surge chamber is a simple special case of the overall analysis.

For system-stability studies Borel chooses the generalised Nyquist theory in preference to Routh's criteria. Complex plane-plotting procedure is outlined to obtain the looped-circuit frequency behaviour, thus defining stability by the $+1$, $+jo$ co-ordinate relationship. Typical diagrams illustrate stable, limiting and unstable surge-chamber and governor systems.

Turbine response to a speed-step function disturbance is a further stage in this analysis, where the general equation is split into three brackets defining guide-vane opening, waterhammer and frequency effects on output variations as a function of time, as well as those influenced by regulating and speed-change-mechanism movements, head, load and voltage.

The fundamental equations for parallel operation are related to those for isolated working and expressions developed for overall mechanical inertia with all voltage change devices blocked and voltage/frequency statism intervening on the voltage regulators. The functional block diagrams for n machines are systematically reduced to establish overall transfer functions and stability criteria using, where appropriate, "equivalent machines" based on hydraulic and mechanical parameters. The procedure is used to ascertain the performance of a single machine in a group of n parallel units subjected to an external disturbance and to confirm analytically that a hydro-electric station on a large system is more stable than when operated separately.

Machine response to speed changes on one unit, and other effects are re-examined and special cases analysed.

A lucid explanation of the gate-opening limiter and its influence on machine performance for rising and falling frequency is coupled with mathematical consideration of steady-state and dynamic behaviour, including step-function disturbance response for grouped machines with some on normal governing.

A new formula for the limiting surge-chamber area derived from the preceding analysis includes damping. Since the governor circuits are faster than those of the surge chamber the problem becomes a special case of the generalised theory, and Routh's stability criterion gives a surge-chamber-stability relationship for isolated operation embracing all installation and

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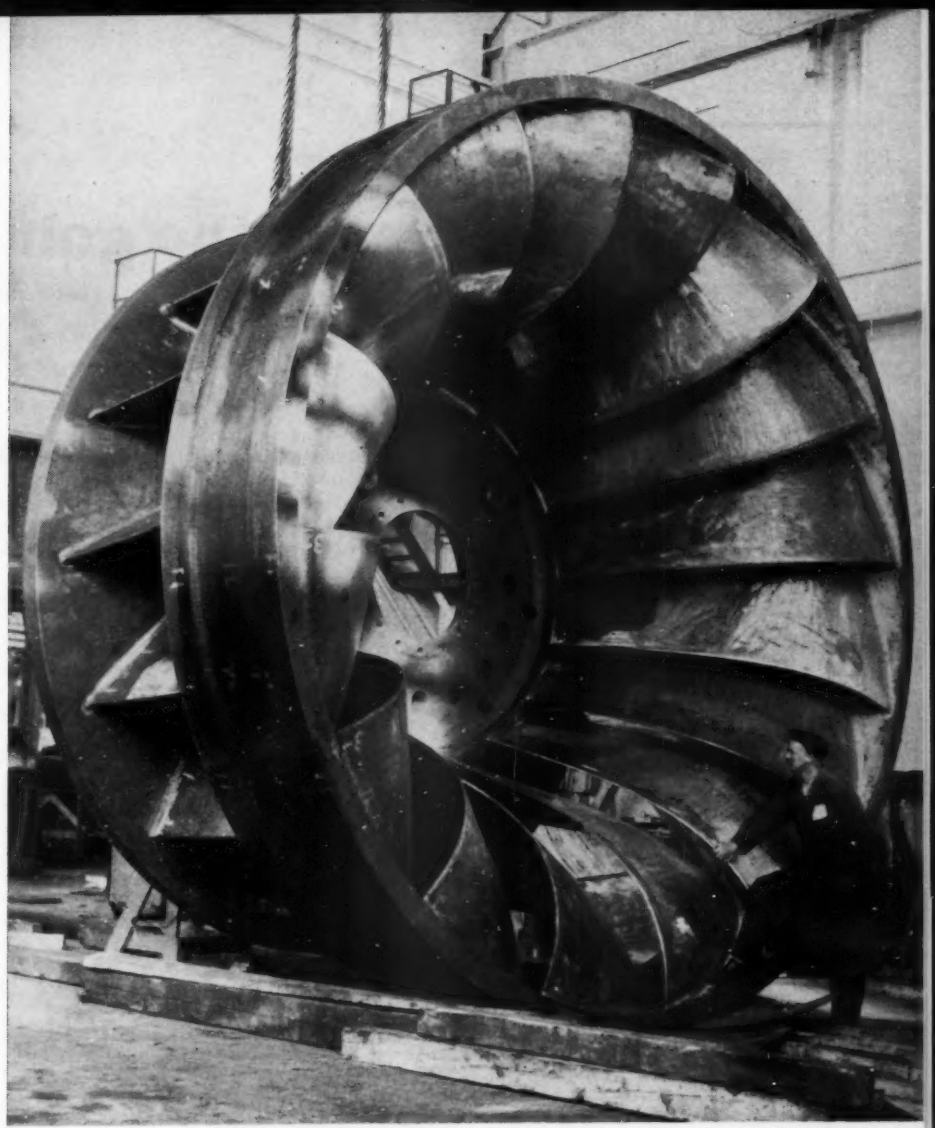

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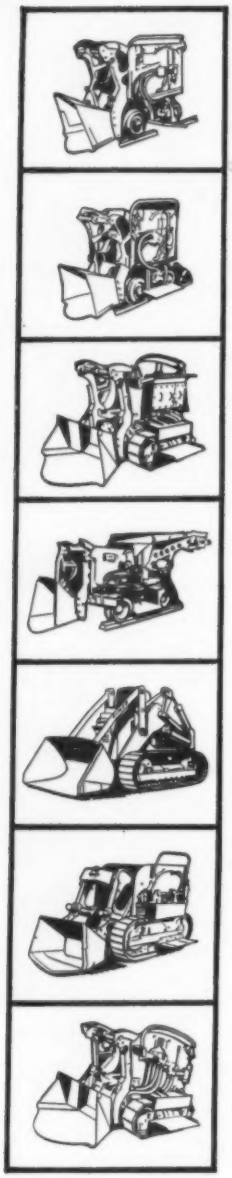

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turbine parameters. Parallel operation is a particular case.

The correlated numerical application of the theoretical treatment covers two systems. The first comprises an isolated system containing five 13·5-MW Francis turbines and a throttled surge chamber. Non-dimensional data and turbine operating parameters, deduced from the contour diagrams, are progressively co-ordinated with system characteristics and the times which relate to: (i) The tachometric servomotor and speed governor circuit (R_1); and (ii) The surge-chamber circuit (C_3); to establish pairs of R_1, C_3 values which define the stability limit. This involves graphic solution of two simultaneous equations. The two resultant curves delineate horizontal and vertical areas where the surge-chamber and governor circuits introduce instability. Any operating points outside these zones is stable.

The Nyquist diagrams can be drawn for given values of R_1 and C_3 if required, and typical examples for stable and unstable governor and surge-chamber combinations demonstrate that overall system behaviour depends on the surge chamber for low pulsations, and on the governor for high pulsations.

Response to a step-function speed-change movement is also calculated to show that transient régimes are conditioned initially by rapidly damped water-hammer and subsequently by the servomotor, and in the steady state by opening speed, statism and turbine-output relationships.

A similar treatment is devoted to a system which includes this station in parallel with another containing six unequally loaded 100-MW units (five regulating and one on gate limit) serving a 480-MW load.

In the example for minimum surge chamber area in isolated operation where the relative Thoma area was 1·00 and the actual value 1·64, the analysis shows that stability only demands 1·27.

Component	Stability factor	Effect on Thoma surge-chamber area (%)
Turbine behaviour	1·86	+86·0
Surge-chamber-junction velocity head	0·69	−31·0
Pipeline head losses	1·07	+ 7·0
Self-regulation	0·92	− 8·0
Overall stability factor ...	1·27	+27·0

English and Continental textbooks have been published recently on hydraulic turbine governors and surge chambers, together with many uncorrelated articles in technical journals. This extensive theoretical literature often lacks the simplicity and generality necessary for overall system operation or to pinpointing components where designs could be improved or relaxed in the interests of performance or economy. The flexible systematic method now proposed utilises the real machine characteristics and elementary numerical procedures which will be welcomed by students and practising engineers engaged on station design or operation.

The excellent printing, clear-cut diagrams and the extensive notation, conveniently folded for reading with the text, makes this original and concise treatise an enjoyable exercise for those with a taste for technical translation and mathematical reasoning.

The treatment might be expanded in future editions to include automatic-voltage-regulator performance, governor-oil hydraulic circuits and the elusive effects of regulating-mechanism inertia, friction, backlash, lost motion, dead band, dead time, and other problems which often bedevil governor performance.

I.E.C. Publication 71. Recommendations for Insulation Co-ordination. Published by the International Electrotechnical Commission, 1, rue de Varembé, Geneva, Switzerland. Third edition 1960. 23 pp. Price 8 Sw. Fr. per copy plus postage.

This publication states the conditions to be fulfilled by the insulation of the various categories of equipment in the form of general recommendations which should, as far as possible, be common to all of them. The recommendations apply to all apparatus for a.c. systems having a nominal voltage greater than 1 kV, for use in exposed installations. They do not apply to generators, rotating machinery, nor to underground cables and overhead lines. The third edition includes a number of additions and amendments to the second edition, which was published in 1958.

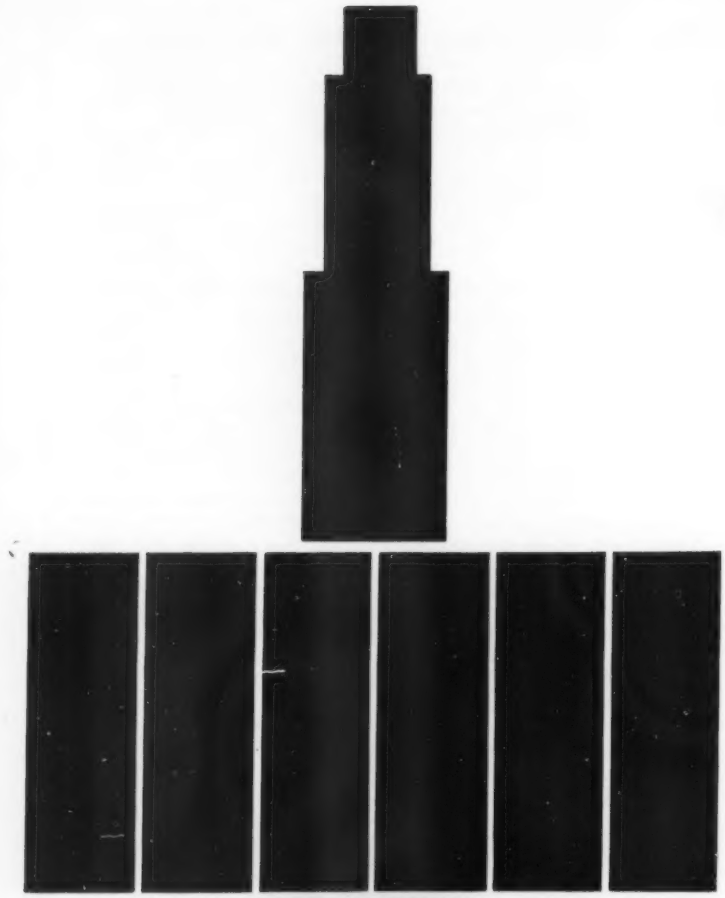
Power from Water. By T. A. L. Paton, C.M.G., B.Sc., M.I.C.E., M.I.Struct.E., F.ASCE., and J. Guthrie Brown, M.I.C.E., P.P.I.Struct.E., M.I.E.Aust. Published by Leonard Hill (Books) Limited, Eden Street, London, N.W.1. 199 pp., 44 ff., 36 plates. 1960. Price 25s. net.

This book is written primarily for the ordinary member of the public who desires a good general description of the essentials of water-power development. However, there can be little doubt that it will also appeal to those professional engineers who are interested in the historical background to hydro-electric work. The authors trace the early days of water power and then devote the bulk of their attention to water-power developments in Great Britain; thus, the book becomes largely a history of the Scottish projects, since Wales has lagged considerably in this field, and England's hydro-electric resources are small.

The authors are comprehensive in their approach, and cover virtually every aspect of the work involved in carrying out a major project, from the preliminary phases through the parliamentary, design, and construction stages to the economics of operation of the completed scheme. Throughout, the language employed is at an intelligent level but is, as far as possible, non-technical, and, probably as a result, the book is extremely readable. Those technical words and phrases which are essential are carefully explained.

The authors rightly emphasise the considerable attention given by hydro-electric engineers to salmon protection and the preservation of amenities. Many examples are cited of how ingenuity has not only overcome the problems posed by these two factors, but has actually resulted in improvements to the tourist and salmon-fishing industries in Scotland. Much of the early strong opposition to the Scottish projects was based on the tenet that the scenic beauty would be jeopardised, and this opposition was often carried to extreme lengths. In the case of the Glen

Power Stations and Substations



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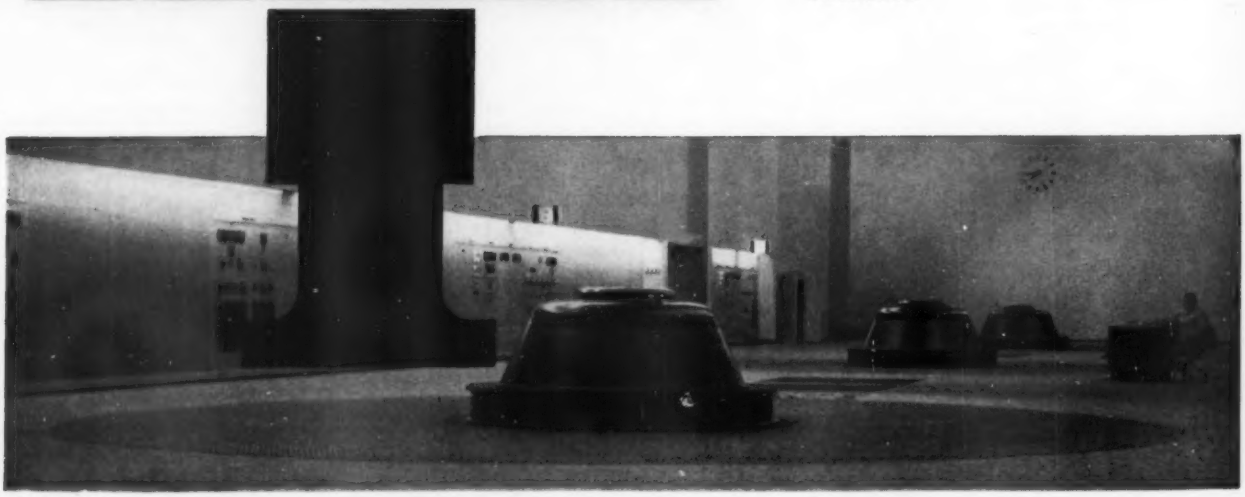

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Affric scheme, the glen was so inaccessible that few visitors were ever able to view its attractions prior to the construction of the access roads for the project.

In addition to hydro-electric developments, the book describes the elements of tidal power, and particularly refers to the Severn Barrage project, with its long history of investigations, reports, and procrastination. The immensity of this scheme still gives cause for admiration but one wonders whether the opportunity to implement the project has not been lost for ever. As the authors suggest, the barrage would probably have been economic if constructed in the thirties, but this might well prove not to be so today.

Apart from a few obvious misprints, particularly in the case of some installed capacities, the book is well presented and the illustrations are excellent. The two appendices list respectively the hydro-electric developments in Great Britain in the period 1896–1937, and those carried out by the North of Scotland Hydro-Electric Board in the period 1948–1959. Brief particulars of catchment, head, installed capacity, and average annual output are given.

The book has a foreword by Lord Reith of Stonehaven, which is written in a distinctive style. We are in complete agreement with him when he says of the authors' account of the development of water power: "A good story they've made of it." It makes good reading for both the technologist and the layman.

Abstracts from the World Technical Press

Rio Sil Basin Development

The Rio Sil Basin is the most important drainage system of north-eastern Spain. The river, which is called Rio Scuro at its source at the Somiedo Col in the Cantabrian Mountains, takes the name of Rio Sil, just after cutting across the Asturias-Leon crest line, until it becomes the Spanish-Portuguese border at Los Peares, 85 km upstream from its mouth, where it is called Rio Miños on the north and Rio Minhos on the south bank. A detailed description is given of the whole development which is divided into four major sections. The Upper Rio Sil (Compañía Hidroeléctrica de Galicia) consists of three plants with a total head of 340 m, a capacity of 145,400 kVA, and an average annual output of 350 million kWh. At the lower part of this section is the Santa María downstream plant, the two upstream stages, Panadrada and Las Ondinas, supplying mainly peak-period energy. The Upper Middle Rio Sil (Empresa Nacional de Electricidad) has four plants which, upon completion of the section, will have a total head of 261·6 m, a capacity of 247,900 h.p., and an average annual output of 522·3 million kWh. The main feature of this section is the 340-million m^3 reservoir created at Barcena by a straight gravity dam 109 m in height, incorporating the power house at its toe. The two downstream stages, Cornatel and Quereño, are still in course of construction, the tailrace of the latter marking the upstream limit of the concession of S.A. Saltos del Sil. The Lower Middle Sil is important on account of the 50% increase in its discharge due to its confluence with its tributary the Rio Ribey. Its seven dams create an aggregate storage of 315 million m^3, corresponding to an output of 157 million kWh. There are eight installations which utilise a total head of 773·9 m, and will have an installed capacity of 510,300 kVA, and an average annual output of 1,718·7 million kWh on completion. Further projects in this section would provide for an additional storage of 345 million m^3, corresponding to 417 million kWh, with an installed capacity of 235,000 kVA and a possible production of 708 million kWh per annum. The development of the Rio Bibey section and its tributaries is shared by two concerns, S.A. Moncabril in the upper, and S.A. Saltos del Sil in the lower stretch. The former's installations, which are either in operation or at the project stage, consist of three on the Rio Bibey itself and three on the Rio Jares. They have a catchment of roughly 750 km^2 and a storage of 185·8 million m^3. They will utilise a total head of 1,336 m, and have a capacity of 143,600 kVA which will produce 412·4 million kWh upon completion of the seven plants of S.A. Saltos del Sil, which are planned to have an aggregate installed capacity of 247,500 kVA and a yearly production of 860·1 million kWh. The Bao gravity dam, briefly described below, is now practically completed. The following data on some of the works of the Saltos del Sil chain are included in the article. The San Martin spillweir intake, at the head of the series, creates a useful storage of 3 million m^3, and is designed for a discharge of 4,000 m^3/sec into the Rio Bibey near Montefurado by means of a 4 km canal. At Montefurado, a gravity dam, 42 m high and 150 m long, provides storage for 7·9 million m^3, and the power station erected at the toe of the dam houses three vertical-shaft Kaplan turbines (two units supplied by Escher-Wyss, Zurich, the third by Neyrpic) operating under a 35-m head at 375 r.p.m., and coupled to 16,000-kVA AEG alternators. The Sequeiros installation downstream from Montefurado, consists of a 300-m diversion canal and a 250-m tunnel connecting through a surge shaft with three penstocks each of which leads to a generating set comprising a vertical-shaft 8,550 h.p. Kaplan turbine of Escher-Wyss make coupled to an ACEC 7,500-kVA alternator. Farther downstream and near the San Estebán there is located the largest plant of the whole basin, with a reservoir extending 43 km upstream. The main features of the San Estebán works include an arch gravity dam 116 m in height and 288 m in crest length, which provides storage of 194·8 million m^3 over an area of 737 hectares, and a crest spillway for a designed discharge of 4,500 m^3/sec through six sector gates, each 15 m by 8 m. Two 6·8-m diameter power tunnels, controlled by 5-m by 8-m caterpillar gates and operating under a head of 68 m, lead from the intake to the power house after bifurcating into steel-lined tunnels, 4·2 m in diameter. The tunnels supply 90,700-h.p. Francis turbines (Karlstads Mekaniska Werkstad), which utilise 75 m^3/sec and operate under a head of 104 m, and the turbines drive 82,500-kVA Metropolitan-Vickers alternators. Immediately upon completion of the San Estebán plant, work began about 8 km downstream on the San Pedro stage. This is a much smaller installation (40,000 kVA),

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which owing to the steepness of the slopes of the gorge at its site presents uncommon features. The four spillway openings, controlled by 15 m by 10 m sector gates, extend over the whole width of the gorge. The power house, which also incorporates the intake, had to be sited upstream from the dam, and the two tailrace tunnels were cut across the bend of the river and discharge below the dam. A short paragraph is devoted to the four-stage series of plant of the Rio Navea, a tributary of the Rio Bibey which it joins just above Montefurado. The main feature of this development is the Chandreja buttress dam, which is 85·5 m high and 240 m long at the crest, with an almost fully utilisable storage of 60,626,000 m³. The article concludes with a description of the Bao gravity dam, which is 110 m in height and has a storage of 238 million m³, corresponding to 245 million kWh. The future underground 311,000-kVA plant at Puente Bibey will be supplied through a 9-km gravity tunnel operating under a head of water of 50 m at maximum storage level. This tunnel will collect the waters of the left-bank Rio Bibey tributary, Rio San Lazaro, through the Manzanada dam and an ancillary tunnel about 1 km long. Driving work on the gravity tunnel began towards the end of 1959 from an adit excavated near the Bao dam. Excavation work for the Puente Bibey machine hall has not yet started, and work on the access roads was scheduled to begin this year. (Georges Vié, *Construction,* Vol. XV, No. 8, August, 1960, p. 293, 11 pp., 17 ff.)

Lower Drava Development

The lower stretch of the Drava, between Ormož and Botovo, presents favourable conditions for power development, and it is estimated that, if combined with a lateral diversion canal on which would be a series of five installations, it has a potential mean annual output of 1,400 GWh. A canal diverting part of the Drava waters into the Sava is also contemplated as an alternative to this project, and these possibilities are now under consideration. The Drava-Sava scheme would utilise seven installations and could produce about 1,600 GWh per annum. This article discusses both schemes and gives an account of the present phase of the preliminary studies. (A. Stepinac, Zagreb Elektroprojekt, *Energija,* Zagreb, Vol. 9, No. 5–6, May–June 1960, p. 141, 7 pp., 11 ff.)

Investigating Scale Effects on Cavitation

When determining the specific speeds and loads of turbines, designers are confronted by cavitation hazards, against which an adequate safety margin must be introduced. The problem is to narrow this safety margin by selecting speeds and loads very close to the threshold of cavitation, and this threshold must be defined and analysed with the greatest accuracy. This kind of investigation can only be carried out by means of small-scale models, but a major problem in this work is presented by the fact that very few of the parameters involved in cavitation can be precisely reproduced in models. The purpose of this paper is to ascertain the most advantageous similarity to be adopted when investigating cavitation in turbines by means of small-scale models, and also to analyse the fluctuations of scale effect in the various degrees of similarity determined by the fluctuation of head scales. This theoretical study shows that scale effect influences absolute pressure, critical pressure, and such factors as the boundary layer and nappe separation, which depend on turbulence. Its results further establish that the Froude similarity is the one associated with the smallest scale effect in the case of horizontal-shaft units, and that it can practically be ignored so long as turbulence in the model is not too small, an effect which is brought about by an excessive reduction in scale. Any remaining scale effects make for greater safety, since cavitation occurs more readily in the model, owing to its restricted head, than in the corresponding full-scale machine. An account is also given of the experimental control tests undertaken at the turbine testing installation of the Chatou laboratory. (J. Beaufrère, *Mémoires & Travaux de la Société Hydrotechnique de France,* No. 1, 1960, p. 51; *La Houille Blanche,* Vol. 15, No. A, July–August, 1960, p. 415; 8 pp., 7 ff.)

Light-Metal Gates and Stoplogs

Not only are stoplogs made of aluminium alloys more and more frequently superseding steel beams, but this trend is also slowly extending to small-span gates. The advance made in this respect is briefly reviewed. Thanks to the improved pressing techniques applied to light-alloy sections, the designer is no longer tied to standard profiles found in manufacturers' price lists, and can now adapt his design to any given conditions and have any type of section made to order. In addition to the fact that light-alloy structures require no protection from corrosion, they are much lighter in weight, and especially in the case of sector gates, further substantial economies can be achieved by the use of light-weight hoisting gears. For instance, the light-alloy sector gate controlling the Vissoie intake of the Navisence power station, 7·2 m wide by 2·7 m useful head, does not exceed 3,300 kg in total weight, fittings included. (Dr. Curt F. Kollbrunner, *Schweizerische Bauzeitung,* Vol. 78, March 3, 1960, pp. 162/163, 3 ff.)

Ferrera Power Station

Continuing the series devoted to the Hinterrhein installations, this article deals particularly with the generating sets and their ancillary equipment of the underground power station at Ferrera. Initially, the project for this plant had been worked out on lines similar to the solution adopted at Bärenberg and Sils, and was to comprise four generating sets driven by Pelton turbines and two storage-pumping sets. The solution ultimately adopted includes three generating-pumping units driven by vertical-shaft Francis turbines, and two relay pumps. The turbines have a maximum discharge of 15·8 m³/sec at a head of 522 m and operate at a speed of 750 r.p.m. The alternators generate at 10·5 kV and have a rated capacity of 70,000 kVA with a power factor of 0·9. The storage pumps deliver 5·2 m³/sec against a head of 499 m and the relay pumps have a rating of 4·0 m³/sec at a head of 41·9 m. The discharge of the pumps when employed as turbines is 4·4 m³/sec and their output is some 1,700 kW. A three-part folder illustrates the layout of the machine hall and the arrangement of the machinery, and shows the longitudinal section and plan of the hall, together with cross sections through a turbine, a generator, a storage pump, and a relay-pumping set, as well as a longitudinal section of a whole generating-pumping set. The three main sets are erected on the same axis in the follow-

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ing order from the south-west to the north-east end of the station: turbine, motor-generator, exciter set, brake system, clutch-coupling, and storage pump. Next to the third set and below the floor of the main hall, a double basement houses the relay pumps and their motor-generators. These reversible relay sets play a major part in the running of the Ferrera plant. Operating as pumps, they can convey excess water from the Innerferrera intake on the Averserrhein to the Sufers storage reservoir while, at the same time, generating energy. On the upstream side, the machine hall, semicircular in cross section, includes the valve chamber which is separated from the main station. The transformers are sited in separate compartments on the downstream side of the hall floor above cells and galleries which house the high-voltage switchgear and the control and high-voltage cables. At a lower level, are accommodated the turbine governors as well as the ancillary equipment of the generating sets. The main control room, an extensive air-conditioning plant, rest room and other accommodation for the personnel, and workshops occupy the north end of the station. (Dipl. El.-Ing. ETH Peter Jaray, *Schweizerische Bauzeitung*, Vol. 78, No. 33, August 18, 1960, p. 535, 4 pp., 1 folding plate, 8 ff.)

Kange River Development

The article describes how this North-Korean scheme progressed from the original Japanese project, partly carried into effect with a single gravity dam between 1937 and 1942, to the present three-stage system of underground plants. The dam, 338 ft high and 1,970 ft long at the crest, was erected at the confluence of the Chan Chin Gan and Sampo Rivers to create a storage reservoir 24,000 million cu ft in capacity, and the whole system is fed from a single intake through a channel 490 ft long. This intake connects with an inclined pressure tunnel leading to a surge tank erected just upstream of a high-speed-gate chamber. From this chamber, the 2,000-ft penstock, which is in a concrete-lined shaft and measures 158 in in diameter at the upper end and 47·4 in at the lower end, trifurcates to supply each of the 47-MW generating sets of the Kange-1 station. The turbine discharge of this plant is conveyed through a gravity tunnel, 3·84 miles in length, to a balancing reservoir from which a 820-ft penstock in shaft leads to the manifold supplying the two 28·2-MW Kange-2 generating sets. The Kange-2 tailwater is discharged into another gravity tunnel, 6·38 miles in length, which connects at its downstream end through a surge tank with the penstocks serving the two 16·8-MW sets at Kange-3. The three stations operate at net heads of 1,310, 522, and 342 ft respectively, the corresponding discharge per unit being 470, 706, and 70 cusecs. The total installed capacity of the system, 231 MW, is fully utilised at the maximum rate of flow of 1,410 cusecs. The total output at the average flow of 1,048 cusecs is 165 MW, and is 100 MW at the lowest guaranteed rate of flow of 623 cusecs. During periods of heavy run-off, excess water is discharged through the spillway block of the dam to the downstream reach of the river. Another dam, downstream of the main one, is at present under consideration and its construction, in conjunction with a pumping plant feeding the main storage reservoir at the head of the whole system, would enable this excess water to be utilised. A downstream extension of the system is also under consideration, and, upon full completion of its development, the Kange River will contribute the major portion of the power requirements of North Korea. A large part of the equipment of the scheme is being supplied by the Blansko works, Moravia, of the ČKD concern. (Dipl. Ing. Zdeněk Pala and Václav Hosnedl, *Czechoslovak Heavy Industry, English Edition*, No. 7, 1960, p. 31, 8 pp., 8 ff.)

NOTE. The above abundantly illustrated issue is entirely devoted to questions of water-power interest, and also contains the following articles: A short editorial note on the Orlík stage of the Vltava system, with an impressive colour photograph of the dam; a description of the East-German Hohenwarte-2 pumped-storage plant, by Dipl. Ing. M. Buchtela and J. Kočí; "Hydraulic and Thermal Conditions on the Sliding Surface of Thrust Bearings in Water Turbines," by Dipl. Ing. Jan Stejskal; "Testing a Turbine for Runaway Speed," by Dipl. Ing. Mirko Vitva; "Thrust Bearings and their Equipment," by Jaroslav Polek. The issue ends with an account of the contribution made by the Czechoslovak engineering industry to Bulgaria's electrification.

Mallero Stream Development

A short description is given of the Sondrio hydro plant which has just been put into service by Società Vizzola at the same time as a new 70-MW unit of their Turbigo thermal plant. The Sondrio station, like the Lanzada upstream plant, forms part of the development of the Mallero basin. The Lanzada discharge and the waters diverted from the Lanterna and Mallero streams collect in a balancing reservoir which feeds a 10-km gravity tunnel discharging 26 m^3/sec. The last two kilometres of this tunnel have a substantially increased cross section, and act as a forebay. The steel penstock, which is housed in an inclined shaft excavated in the rock and is 1 km in total length, measures 3·1 m in diameter at the top, and 2·2 m at its downstream end. The underground power house is accessible through a tunnel from the Stelvio highway. The station has been operating since last April with a vertical-shaft 75,000-kVA Ansaldo San-Giorgio alternator driven by a Pelton wheel of Ansaldo design, and will ultimately house another identical generating set now in course of construction. The discharge of the plant is conveyed to the Adda River through a tailrace 1·5 km long which consists of a tunnel and a syphon connected with an open channel. The outdoor substation and step-up transformers are sited near the entrance of the access tunnel. The total yearly production is assessed at 408 million kWh. (*Notizie IRI*, Instituto per la Ricostruzione Industriale, Roma, No. 18, June–July, 1960, p. 12, 1 f.)

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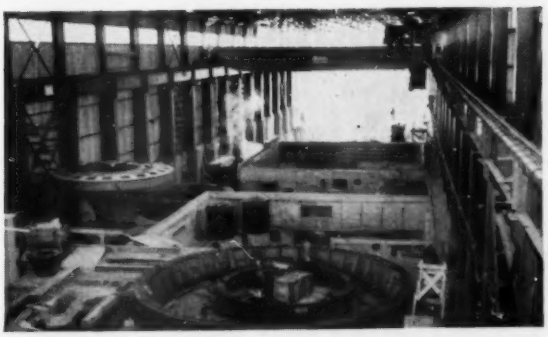
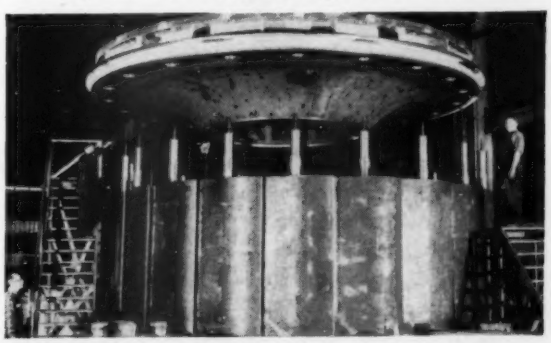
Kelsey Power House during construction.*



Power House Interior during construction.*

Runner, Hub and Head Cover prior to installation.*



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The generator presents a high-torsoed appearance, having been so designed in consideration of the comparatively high speed. Since the generator was designed for a rated voltage of 16,500 volts and, in this respect, represents the first such attempt in Japan, particular care was taken in the corona-shielding and impulse resistance of the stator winding. The generator is rated at 60 cycles, 225 r.p.m. and 0.9 power factor, but is also capable of operating at 50 cycles, 187.5 r.p.m. and 1.0 power factor.

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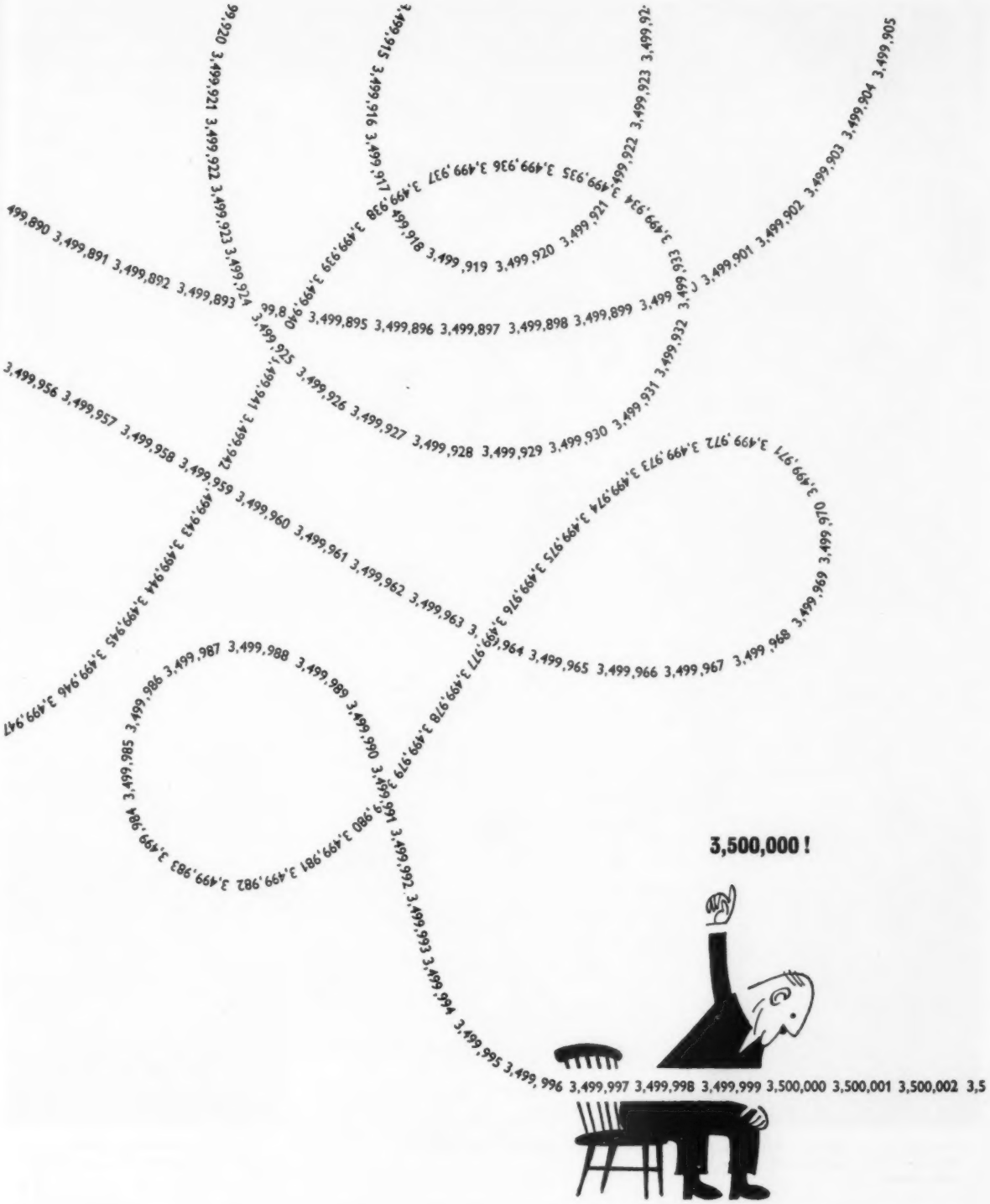

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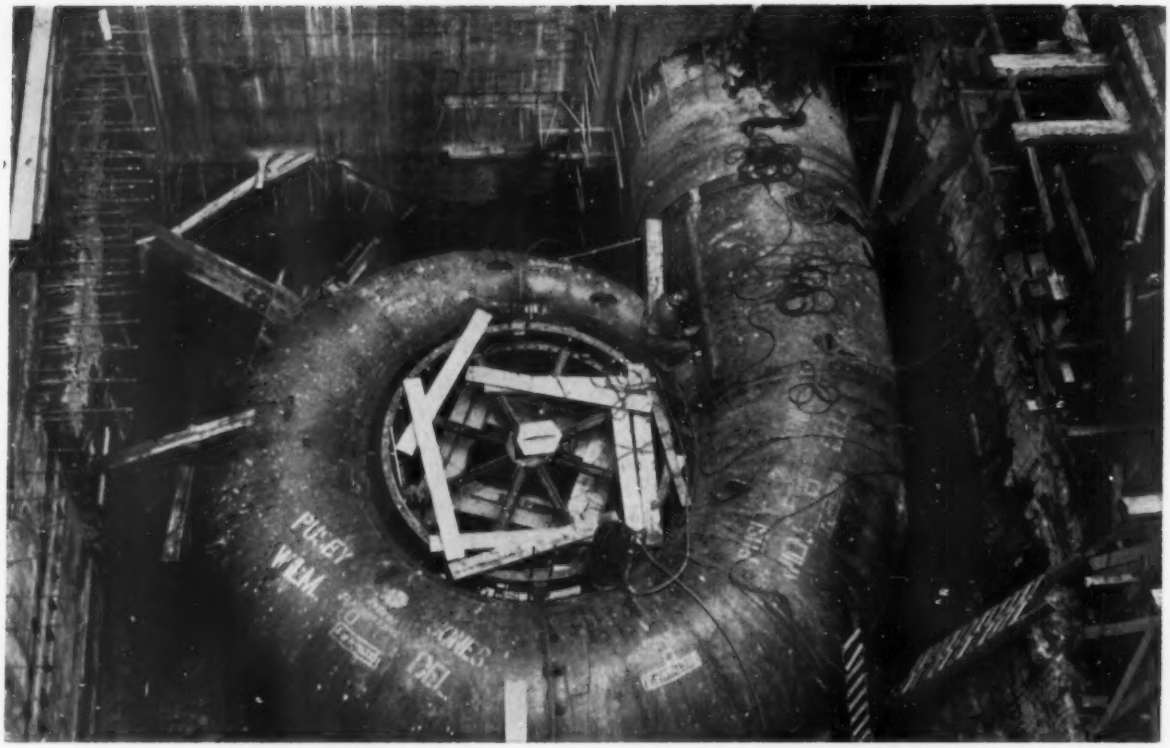

Plate steel spiral scroll case during installation at the Prospect powerhouse.

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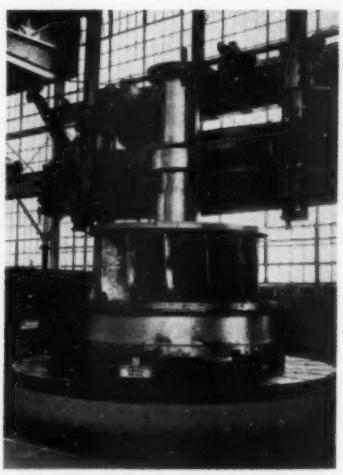

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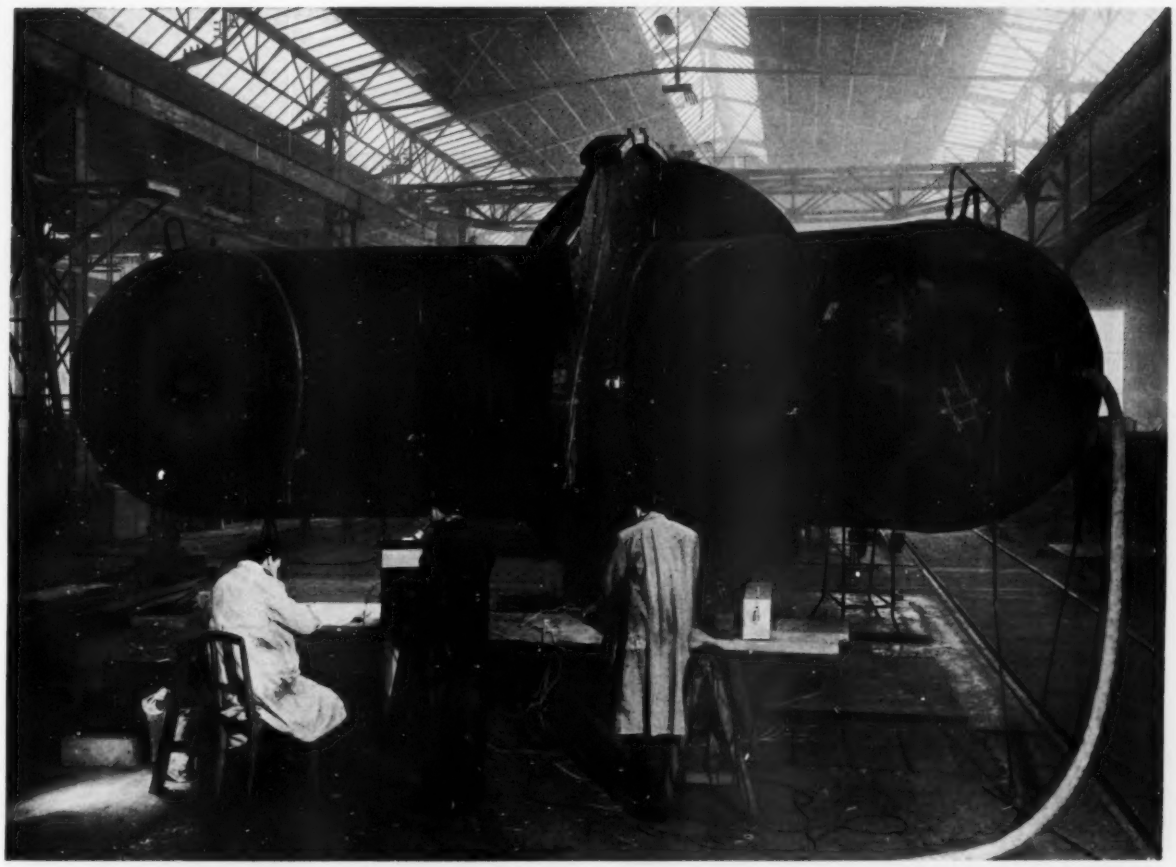

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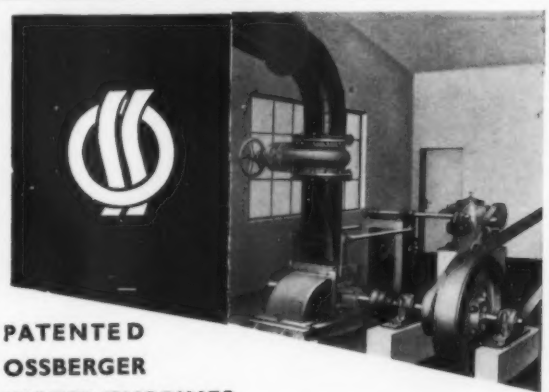

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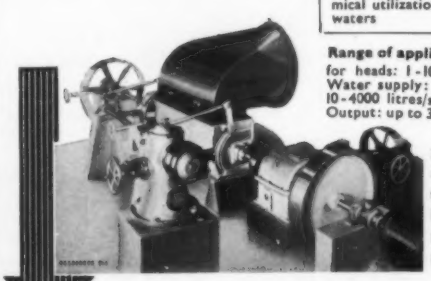

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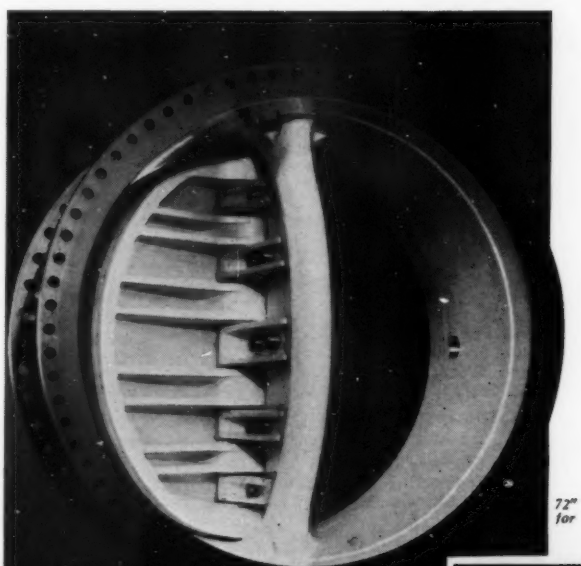

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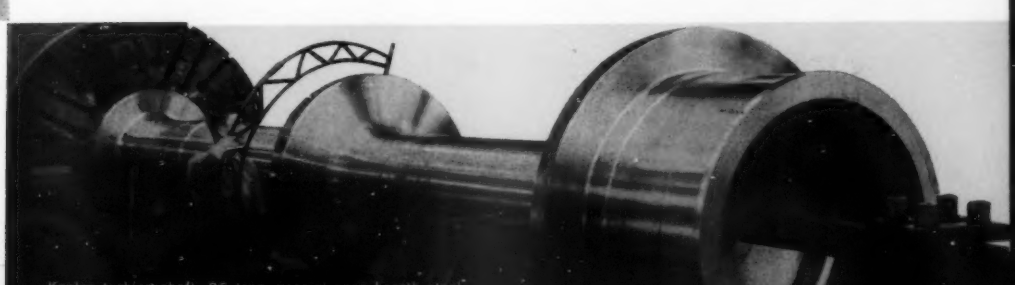

Kaplan turbine shaft, 26 tons, special open hearth steel

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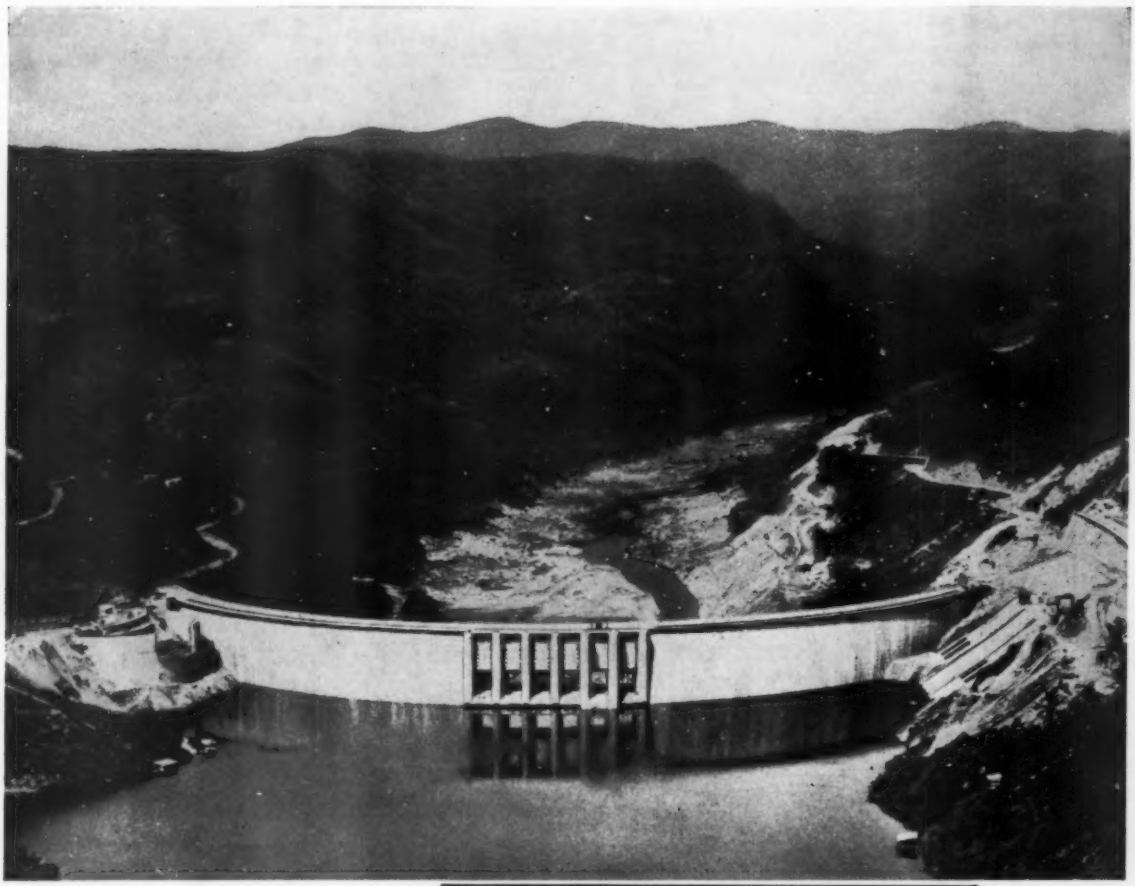

Consulting Electrical and Mechanical Engineers: Messrs. Merz and McLellan

and AEI

Six 100-MW generators for Stage One of the massive hydro-electric project at Kariba, on the Zambesi river, have been supplied by AEI. They are the largest hydro-electric generators in the entire continent of Africa.

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